

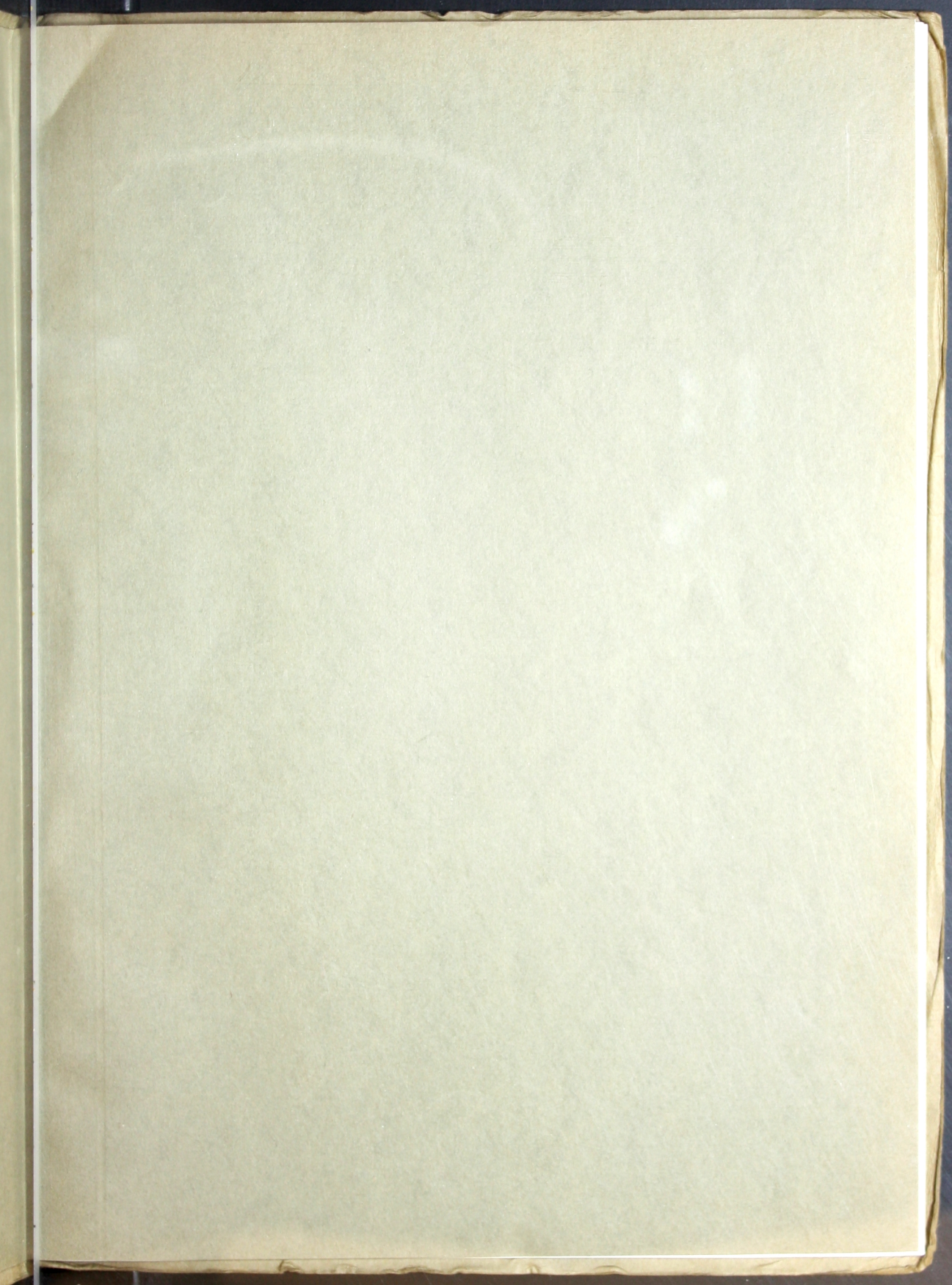
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OXWELDED ROOF TRUSSES

By H. H. MOSS



THE LINDE AIR PRODUCTS COMPANY



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Oxwelded Roof Trusses

A Study of Insert Plate Joints in FINK TYPE ROOF TRUSSES

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Contents

The arrangement of the data in this publication follows chronological order as much as is possible. It is grouped as follows under the headings of:

	PAGE
Introduction	3
Preliminary Studies	4
Design Assumptions for Stress Transfer by the Insert Plate Method	6
Application of Insert Plate to Practical Truss Structures	9
Fabrication of Welded Trusses	33
Testing Structure and Methods	39
General Summary of Results	47

To the several parts of the appendix are reserved the function of making comparisons between the welded and the riveted construction and between the various factors of the welded construction as these were developed from truss to truss. The arrangement of the appendix follows the outline given below:

	PAGE
APPENDIX A—Investigation of Stress Distribution . .	51
APPENDIX B—Comparison of Factors of Safety and of Deflections	59
APPENDIX C—Deflection Graphs	60
APPENDIX D—Time Data for Fabrication of Trusses .	61
APPENDIX E—Cost Studies	63
APPENDIX F—Physical Properties of Truss Material .	69

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Introduction

THIS publication consists of a report of the design, development, fabrication and testing of a series of oxy-acetylene welded roof trusses of the Fink type. This work begins a program by The Linde Air Products Company looking towards the practical utilization of the oxy-acetylene process in the fabrication of structural steel.

It now makes available to consulting engineers engaged in structural and welding work, and to engineers, superintendents and other officials of steel fabricating plants, information on the design, fabrication procedure, and ultimate strength of a series of 40-foot roof trusses. At the beginning none of these factors were fixed. Each subsequent truss designed and tested (there were five) had to its advantage the information gained on the preceding ones, so that the final results and procedure can be said to be well founded.

Probably the most important development of the work was the method of joining structural steel which causes gusset plates to be inserted and made a part of the truss members by butt welding. It enables a stronger truss to be made with less steel and less labor. As the development of this type of connection indicates, the whole work was approached from the standpoint that a new method of joining, such as welding, would require a new type of joint, rather than an adaptation of the conventional riveted construction.

The Linde Air Products Company is glad to be able to pioneer in this line of investigation. While it has many men in its organization who know far more about the characteristics of the welded joint in steel than they know about structural engineering, it was nevertheless necessary for the leading interest in the oxy-acetylene industry to blaze the way in this new application. Lest it be thought presumptuous on their part to enter this new field, it may be said that the same men in this organization, without being pipe-line engineers, have in less than ten years promoted the welded pipe joint to a place where it is standard for long trunk lines carrying petroleum; without being refrigerating engineers, they have introduced oxwelded construction into all domestic refrigerators; without being coach builders or cabinet-makers, they have convinced the builders of highest grade auto bodies and metal furniture that the oxwelded joint fits their requirements best.

Progressive structural engineers, designers and erectors can not and will not shut their eyes to the possibilities which their brother engineers have found in the oxy-acetylene weld as a method of joining steel. To such men we issue a cordial invitation to allow us to place at their disposal the facts we have accumulated on this new joint. With this information they will be in a position to utilize this process in a way which best conforms to the requirements of their individual problems.

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Preliminary Studies

AT the time of starting this work, there was very little design data available on the welding of steel roof trusses. Such that was at hand pertained almost entirely to untested field applications. The subject matter of this report was therefore developed on an empirical basis of which the following preliminary studies of welded joints constituted the first step.

The oxy-acetylene welding engineer prefers the simple butt weld and utilizes this means of joining his metal whenever possible. Two of the most dependable factors in butt welding consist of its ease of analysis from a standpoint of strength and its characteristics from an inspection standpoint in lending itself splendidly to visual and manual inspection. With the commercial advent of high test or high quality steel welding rods, it has been consistently demonstrated that butt welds in structural steel, with but slight reinforcement, will develop a strength exceeding that of the so-called "base metal."

Butt welding principles conflicted with the principle of joint design of riveted structures. In view of this, means had to be devised to maintain a single plane of metal at a joint where the vertical legs of

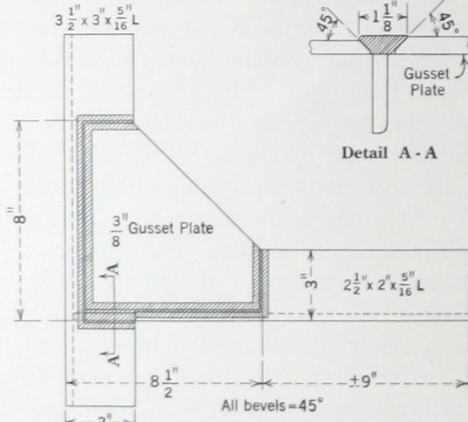


FIG. 1. The insert plate as applied to the specimen joints in the original pilot tests



FIG. 2. Typical fracture of specimen shown in Fig. 1, resulting from a tensile test

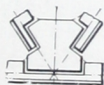
the members connected with the gusset plates. The joint shown in Fig. 1 utilizes this principle in joining two structural angles by means of gusset plate inserts, the legs of both angles having been removed by the aid of a cutting blowpipe. An hypothesis for designing the joint is given below.

A number of full-size specimens, similar to Fig. 1, were made up in The Union

Carbide and Carbon Research Laboratories and subjected to a tensile test in a 100,000-lb. capacity Olsen tensile machine. The specimens were welded with Oxweld High Test steel welding rod, which has a normal tensile strength of 58,000 lb. per square inch, *in the weld*, and a yield point of approximately 42,000 lb. per square inch.

Failure occurred in the base metal below the lower tension weld and weld zone as shown typically in Fig. 2. The average tensile strength per square inch for three specimens was 55,600 lb. A typical specimen gauged for elongation recorded 12 per cent in 2 inches over the tension weld in the angle iron.

These tests indicated that the full strength of an angle could be transferred to other parts by the insert plate method of joining, and, further, that symmetry of the section would undoubtedly permit closer adherence to theoretical plate requirements.



Design Assumptions for Stress Transfer by Insert Plate Method

HAVING as a background the results of these laboratory tests, the next step consisted in applying this principle of joining to a practical truss structure. This not only necessitated a study of means and ways of fashioning the insert plate to various types of joints, but also a working design procedure for the proportionment of the insert plate to properly transfer the stresses functioning at a particular joint.

The insert plate method of joining structural shapes provides a means of

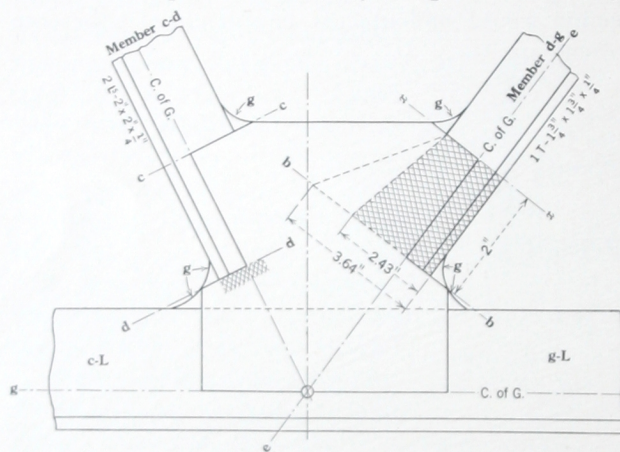


FIG. 3. Principle of the insert plate as applied to a joint in a Fink type truss

disposing the represented strength of structural shapes into single common planes. If symmetrical shapes or members are selected, such as tees, angles back to back, or I-sections, the webbing or gusset plates will lie in a plane coinciding with webs or vertical legs of the members so joined, and by increasing the thickness of these webs or gusset plates over that of the member metal being joined, their dimensions are held within practical limits. By placing the welded joint on the center of gravity plane in a body of metal lying symmetrical with the plane of stress action, direct stress transportation from members to gusset plates is obtained, or nearly so.

An illustration of the insert plate as applied to a joint in a Fink truss is given in Fig. 3. Members *c-L* and *g-L* represent the lower chord, member *c-d* the main strut, and member *d-g* is a diagonal web tension member.

The following hypothesis for joint design is given only as a working expression of the joint stress transfer theory until further study and experimental work along these lines is conducted. The tensional stress of member *d-g* is transferred to the gusset plate primarily by the shear weld along the center of gravity line *e-e*. The amount of stress to be transferred by this shear

members so joined, and by increasing the thickness of these webs or gusset plates over that of the member metal being joined, their dimensions are held within practical limits. By placing the welded joint on the center of gravity plane in a body of metal lying symmetrical with the plane of stress action, direct stress transportation from members to gusset plates is obtained, or nearly so.

weld is taken to be the difference between the whole stress in the member $d-g$ and that portion of stress transferred by the vertical leg above $e-e$, welded to the edge of the gusset plate in the line $x-x$, called the "tension weld." In other words, the short butt weld at the line $x-x$, terminating in the fillet g , is calculated to take care of the metal in member $d-g$ above the center of the gravity line $e-e$. When designing the joint, the whole strength of the tension member should be used rather than the indicated diagram stress which may be lower. The strength of the butt weld along the line $b-b$ is disregarded when determining the shear weld, even though it does function to join the member to the gusset plate and to transfer a part of the member stress. If the remaining stress to be transferred is divided by the unit shearing strength of a weld, either working or maximum, the length of the shear weld along the center of gravity line $e-e$ will be determined.

It is not held that this hypothesis, which serves as a basis for designing the insert plate joints for the welded trusses, is representative of the true conditions of stress transfer or stress disposition within the plate. It, however, is assumed that the stress areas of the plate concentrate more or less along the center of gravity planes at the origin of stress transfer from the members, and diverge toward their component areas as they progress into the plate. On this assumption the true stress areas are, therefore, not exactly as shown in the sketch, but do involve these areas within their extreme boundaries.

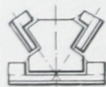
The same procedure can be followed to obtain the dimension of the shear weld for the compression member $c-d$, but due to the low unit stress in this member (a column), it is not necessary to develop its full strength. For the joint of the compression members in the experimental cases, the indicated diagram stress was used and 2 in. placed as the minimum shear weld length. The fillets g are again used to prevent abrupt angles and to strengthen the weld terminals.

In designing the insert plate, the principle is observed that the stress in a structural tee member will transfer, by the method shown, to a single element of metal lying in a plane symmetrical with respect to the leg of the tee. The length of the base of this element (at $b-b$) is seen to be 2.43 in., where the area at the base is equal to the area of the member. This product is the area 0.91 sq. in. divided by .375 (the thickness of the insert plate). When proportioning the insert plate, the lower butt joint, welded at $b-b$, is taken into account as the metal is continuous across this joint. With a $\frac{1}{4}$ -in. plate, under the same conditions, the base length is 3.64 in. The latter, while still within the plate zone to the right of the vertical center line of the plate, is not as well placed with respect to the center of gravity line $e-e$ of the member.

For the compression member, a considerably smaller plate area at the

base line $d-d$ is required, as the stress that this member as a column imparts to the plate is small compared to its strength immediately at the joint. By taking the indicated diagram stress of 5400 lb. and 15,000 lb. per square inch for unit working compression strength of the plate at this section, the base length will be less than one inch. The design provides in excess of this length, thereby reducing the unit stress in the plate.

Attachment to the lower chord is obtained by inserting the plate into the vertical leg of the lower chord member to the center of gravity line $g-g$, as shown. The plate being thicker than the vertical leg, the lower chord is made stronger at this point. The component stresses in members $d-g$ and $c-d$ acting horizontally and to the right of the point x are thus amply provided for. However, the lower chord joint design can be checked by the methods used for the joint in the tension member $d-g$.



Application of Insert Plate to a Practical Truss Structure

UPON the foregoing hypothesis of joint design, the trusses for the test program were worked out. In all, five welded trusses were designed, fabricated, and tested to failure. For purposes of strength comparison, an order was placed with a leading steel fabricator for a riveted truss conforming to the same specifications as used for the welded trusses. The trusses are designated as Cases 1, 2, 3, 4, 5 and 6. Case 2 is the riveted truss, and Cases 1, 3, 4, 5 and 6 were developed in the order named. Conservative working factors were used for all except Case 6, where the yield point of the base metal was the basis for the unit strength design factors.

Compound Fink type trusses were selected, having the following characteristics:

Span, 40 feet.	Spacing of trusses, 15 feet <i>C-C</i> .
Pitch, one-quarter.	Number of panels, eight.
Total load per square foot of horizontal projection.....	40 lb.
Giving a vertical, panel or working load of.....	3000 lb.
Design of truss to meet the requirements of strength of the New York City Building Code.	

The first truss design (Case 1), utilizing the insert plate, took into consideration many factors that were later found to have little or no importance in regard to the strength of a welded truss. These factors operated to lower the unit working strength of the several metals involved. This initial work served as an example of precedent for the subsequent designs.

In the following few pages the more important design calculations will be considered. The set of six drawing plates provides a comparison between the welded trusses and the riveted truss as well as showing the progressive development of the insert joint. The design of the first welded truss (Case 1) is rather completely worked out as is the design of the pipe truss. The important changes made in the other designs as dictated by experience are pointed out. Comparisons of the welded and riveted construction on the basis of cost, stress distribution, developed factors of safety, and deflections are made in the appendix.

CASE 1

A WELDED TRUSS composed of angles, tees and plates. Primarily a pre-experimental case for general observation purposes, and to test the practicability of the testing structure, later described.

DESIGN ASSUMPTION AND ANALYSIS

Having no precedents or practical experience with a composite welded structure employing the insert plate joint design, it was decided to use very conservative working stresses for both member and joint designs, thus providing a structure uniformly over-designed. Under this condition, the several types of insert plate joints could be observed and studied at loads well above the normal designed strength of the structure, provided that welding would not introduce factors that would lower the strength of the members. Attention was given to the possible effect of residual weld stresses on the member and the so-called "fusion zone effect." Slight allowances were made over theoretical in determining the length of welds, which were to be made in a shop not heretofore experienced with special welding rods.

The following working stresses were used for base and weld metal for Case 1:

	Lb. per sq. in.
Tensile strength	13,750
Shear strength	9,000
Compression (for short prisms)	15,000
Compression (for rafters and struts)	15,000—70 _R ^L

Thickness of all welds was taken as the thickness of plates welded. Where two plates of varying thicknesses were welded together, the thickness of the lighter plate governed.

The design procedure for a truss or similar structure embraces two methods, viz., member design and joint design. The allowable compressive strength in the members was compared to the indicated diagram stress + 10 per cent. (The 10 per cent was added to provide additional metal to offset any residual stress that might be introduced as a result of the welding operation.) The procedure for joint design for this case endeavors to develop the full strength of both tension and compression members joined, as contrasted with the usual procedure which uses the indicated diagram stress. Invariably the above method will give a higher joint strength than the diagram strength prescribes, due to oversize areas of the members selected. The plate insert joint causes the welds at the terminals of each member to function in shear and compression, or shear and tension. The proportionment of these couples is made by deducting the effective abutting strength,

functioning either in tension or compression, from the total tensile or compressive strength to the member determined by the preceding data, the result being the total shearing strength that must be transferred to the gusset plate.

Reference to Plate I should be made in studying the following member and joint design calculations.

DESIGN OF MEMBERS

Compression Members

A-a diagram stress = 23,500 lb. (Controls)

B-b diagram stress = 22,100 lb.

C-e diagram stress = 20,800 lb.

D-f diagram stress = 19,400 lb.

Member *A-a*

Working stress $23,500 \times 1.10 = 25,800$ lb.

Angles selected, two 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ in., safe load = 26,600 lb.

Next smaller 2, 3 by 2 by $\frac{1}{4}$ in., safe load = 20,600 lb.

$L = 67$ in.

$R = 0.95$ and 1.00 (in contact) back to back.

$a = 2.64$ sq. in. for 2, 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ in. angles.

$f_c = 15,000 - 70 \times 67/.95 = 10,065$.

$f_{ca} = 10,065 \times 2.64 = 26,600$ in.

Member *c-d*

Diagram stress = 5400 lb.

Working stress = $5400 \times 1.10 = 5940$ lb.

Assembly provided tees (back to back).

$L = 67$ in. ($L/r = 120$ max.).

Therefore $r = 67/120 = .56$.

r for 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ in. $T = .73$ and $.65$.

$f_c = 15,000 - 70 \times 67/.65 = 7800$.

$a = 1.77$.

$f_{ca} = 1.77 \times 7800 = 13,800 \times 2 = 27,600$ lb.

This is greatly in excess of the indicated stress. However, were used for purposes of symmetry.

Members *a-b* and *e-f*

a-b diagram stress = 2700 lb.

e-f diagram stress = 2700 lb.

Working stress = $2700 \times 1.10 = 2970$ lb.; say 3000 lb.

$L = 34$ in.

As $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{4}$ in. angles were assumed

to be the smallest shapes that afforded adequate metal at the welded joint, these will be tried.

r for about angles = 0.45 , $L/r = 34/.45 = 75.5$.

$A = 0.69 f = 15,000 - 70 \times 75.5 = 9715$ lb.

$f_{ca} = 9715 \times 0.69 = 6700 \times 2 = 13,400$ lb. which is much in excess of working stress.

Tension Members

a-L diagram stress = 21,000 lb. (Controls)

c-L diagram stress = 18,000 lb.

g-L diagram stress = 12,000 lb.

$a = \frac{21,000}{13,750} = 1.53$ sq. in. for 2 angles.

Area of two 2 by 2 by $\frac{1}{4}$ in. angles = 1.88 sq. in. (nearest stock size).

$1.88 \times 13,750 = 25,800$ lb. plus.

Member *f-g*

Diagram stress = 9000 lb.

Member *d-g*

Diagram stress = 6000 lb.

$a = \frac{9000}{13,750} = 0.66$.

a of 2, $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{4}$ in. angles = 1.38 (minimum size used).

$1.38 \times 13,750 = 19,000$ lb.

Member *b-c*

Diagram stress = 3000 lb.

Member *e-d*

Diagram stress = 3000 lb.

$a = \frac{3000}{13,750} = 0.22$ sq. in.

Minimum size used = $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{4}$ in.

$a = 1.28$.

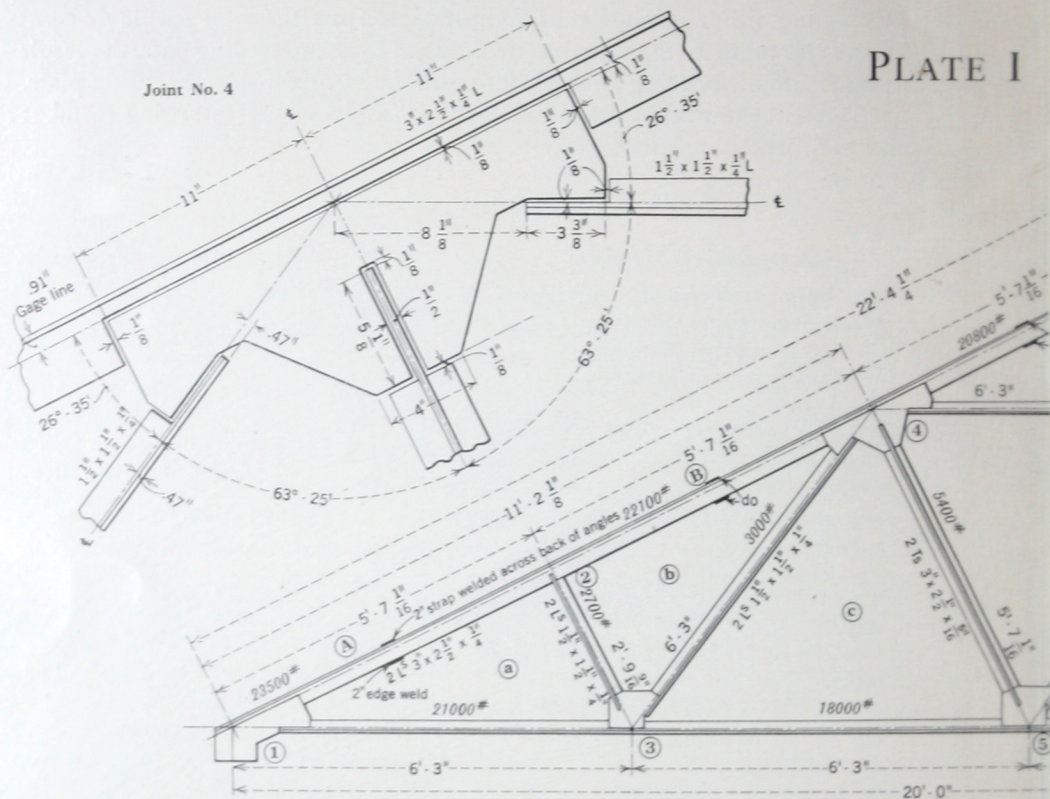
$f_{ca} = 19,000$ lb.

Member *g-g*

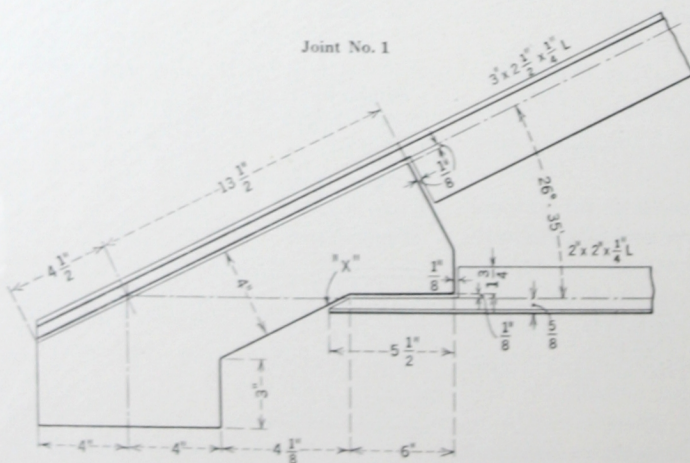
Diagram stress = 0.

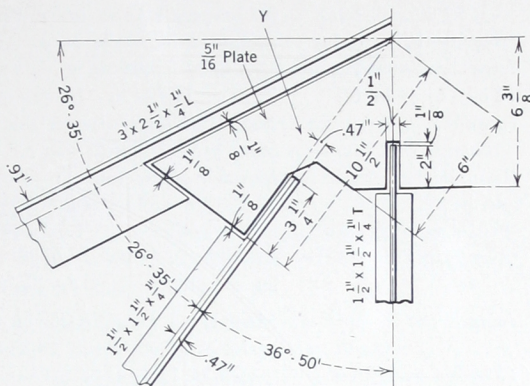
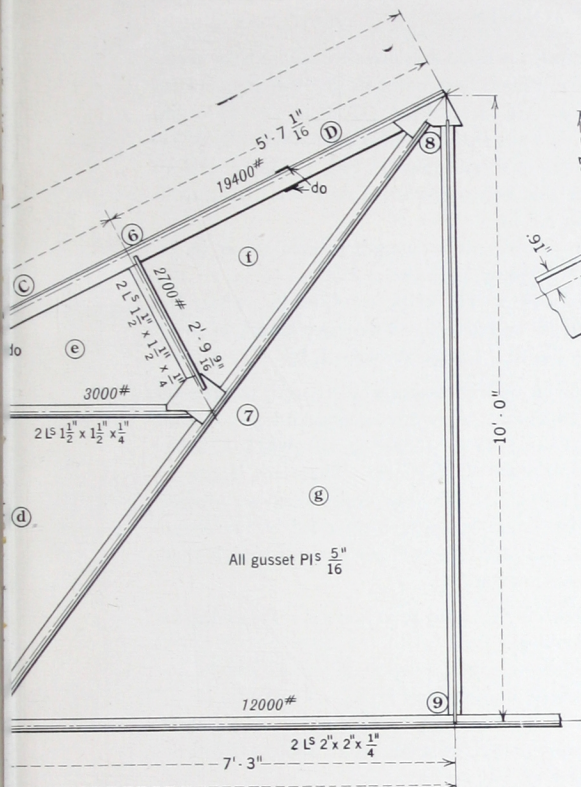
2, $1\frac{1}{2}$ by $1\frac{1}{2}$ by $\frac{1}{4}$ in. tees will be used.

PLATE I



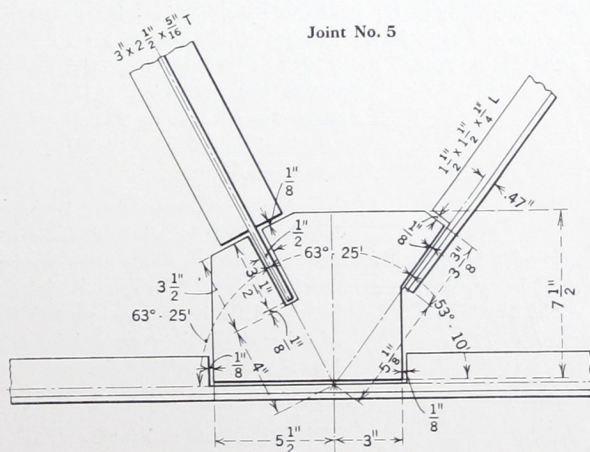
Joint No. 1





Joint No. 8

PLATE I. This plate is a drawing of the first application of the insert plate as applied to a Fink type truss, which furnished experimental data for the subsequent truss designs; also a full scale means of determining the practicability of the testing structure



DESIGN OF JOINTS

Joint 1

(3 by 2½ by ¼ in. L)
(gage line ⅞ in.)
(2 by 2 by ¼ in. L)
(gage line ⅝ in.)

Member *A-a* (compression): Total compression strength of member = area 1.31 sq. in. $\times$ 15,000 lb. = 19,650 lb. Compression leg area = 3 in. $- \frac{7}{8}$ in. = 2½ in. $(.125) \times .25$ in. = .53 sq. in. $\times$ 15,000 lb. per sq. in. = 7950 lb. Shear leg strength = 19,650 $-$ 7950 = 11,700 lb. Length of shear leg = 11,700 $\div$ 2250 = 5.2 in. Nature of joint requires this leg to extend to eave line resulting in much greater length than 5.2 in.

Member *a-L* (tension): Total tensile strength = area .94 sq. in. $\times$ 13,750 lb. = 12,800 lb. Tension leg area = 2 in. $- \frac{5}{8}$ in. = 1⅜ in. = 1.375 $\times$.25 = .3444 $\times$ 13,750 lb. = 4730 lb. Shear leg strength = 12,800 $-$ 4730 = 8070 lb. Therefore length of shear leg = 8070 $\div$ 2250 = 3.6 in. Design provides 6 in.

Joint 2

(3 by 2½ by ¼ in. L)
(1½ by 1½ by ¼ in. L)

Member *a-b* (compression): Total compression strength = area .69 sq. in. $\times$ 15,000 lb. = 10,350 lb. Area of compression leg = 1.5 in. $- .25$ = 1.25 in. $\times$.25 = .3125 $\times$ 15,000 lb. = 4680 lb. Shear leg strength = 10,350 $-$ 4680 = 5670 lb. Therefore length of shear leg = 5670 $\div$ 2250 = 2.52 in. The design does not provide this length, but due to the excess of strength provided by the design over the indicated stress, the shear leg will not be extended beyond the gage line of the upper chord angles.

Joint 3

(2 by 2 by ¼ in. L)
(1½ by 1½ by ¼ in. L)
(1½ by 1½ by ¼ in. L)
(gage line ½ in.)

Member *a-b* (compression) is the same as in joint 2. Shear leg = 2.52 in. Design provides 3 in.

Member *b-c*: Total tensile strength = area .69 $\times$ 13,750 lb. = 9500 lb. plus or minus. Area tensile leg = 1.50 $- .50$ = 1.00 in. $\times$.25 = .25 $\times$ 13,750 lb. = 3440 lb. Shear leg strength = 9500 $-$ 3440 = 6060 lb. Therefore length of shear leg = 6060 $\div$ 2250 = 2.7 in. Design provides 4 in.

Joint 4

(3 by 2½ by ¼ in. L)
(1½ by 1½ by ¼ in. L)
(3 by 2½ by ⅝ in. T)

Member *b-c* (tension) is the same as joint 3. Shear leg = 2.7 in. Design provides 4 in.

Member *d-e* (tension) is the same as joint 3.

Member *C-d* (compression): Total compression strength = area 1.77 $\times$ 15,000 lb. = 26,600 lb. Area compression leg = 3.00 in. $- .3125$ = 2.6875 in. $\times$.3125 = .84 sq. in. $\times$ 15,000 = 12,600 lb. Shear leg strength = 26,600 $-$ 12,600 = 14,000 lb. Therefore, length of shear leg = 14,000 $\div$ (9000 $\times$.3125) = 5 in. Design provides 5 in.

Joint 5

(3 by 2½ by ⅝ in. T)
(1½ by 1½ by ¼ in. L)
(2 by 2 by ¼ in. L)

Member *c-d* (compression) is the same as joint 4, requiring 5 in. length of shear leg. The design provides only 3½ in., which is more than sufficient to transfer the maximum probable stress that will be developed in member *c-d*. At joint 4 the design length of 5 in. has been used for the purpose of stiffening the wide gusset plates.

Member *d-g* is the same as joint 3, requiring 2.7 in. length of shear leg. Design provides 4 in.

Joint 6

This is the same as joint 2.

Joint 7

This is the same as joint 3.

Joint 8

(3 by 2½ by ¼ in. L)
(1½ by 1½ by ¼ in. L)
(1½ by 1½ by ¼ in. T)

Member *f-g* (tension) is the same as Member *b-c*. Joint 3: 2.7 in. required, 4 in. provided. Member *g-x* (no indicated stress). Design provides 2 in. shear leg = 2250 lb. $\times$ 2 in. = 4500 lb. and (1.5 in. $\times$.25 in.) $\times$.25 $\times$ 13,750 lb. = 4300 lb. tension. The sum of 4500 lb. and 4300 lb. = 8800 lb. for each tee. 8800 lb. $\times$ 2 = 17,600 lb.

Member *D-f* (compression): Total strength of member = 19,650 lb. (see joint 1). The abutting edge in this case is no doubt in tension under maximum load in the event of elongation in the bottom chord angles. Therefore tension leg metal = 2.125 $\times$.25 = .53 sq. in. $\times$ 13,750 lb. = 7280 lb. Shear portion = 19,650 - 7280 = 12,370 lb. Therefore length of shear leg = 12,370 $\div$ 2250 = 5½ in. plus or minus. Design provides 11¾ in.

Joint 9

(2 by 2 by ¼ in. L)

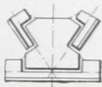
In addition to the securement of the 1½ by 1½ by ¼ in. tee, a splice will have to be made in the lower chord angles at this point.

Member *g-x* (no indicated stress): Design provides (1.50 - .25) $\times$.25 $\times$ 13,750 plus (1.375 $\times$.25 $\times$ 2250) = 5070 lb., total tensile strength per tee $\times$ 2 = 10,140 lb.

Splice (tension): Total strength of member = area .94 $\times$ 13,750 lb. = 12,900 lb. $\div$ 2250 = 5¾ in. plus or minus. 6 in. will be used. The preparation of splice for the two lower chord angles, to alternate the tension butt weld.

Purlin Clips

5 by 3½ by ⅝ in. purlin clips to be welded to the top chord angles, the 5 in. leg against the 8 in. channel purlin. The 2½ in. chord angles to be drilled for one ¾ in. purlin flange bolt. Maximum shear in clip angles = parallel component of panel load = 1400 lb. Design provides 3.5 $\times$ 2 = 7 in. $\times$ 1685 lb. = 11,800 lb. For a permanent structure the bolts specified above to secure the channel flange to the top chord angles, would not be used. The channel flange would be welded to a small strap clip which, in turn, would weld to the out-board edges of the chord angles, this avoiding any reduction in truss metal area.



CASE 2

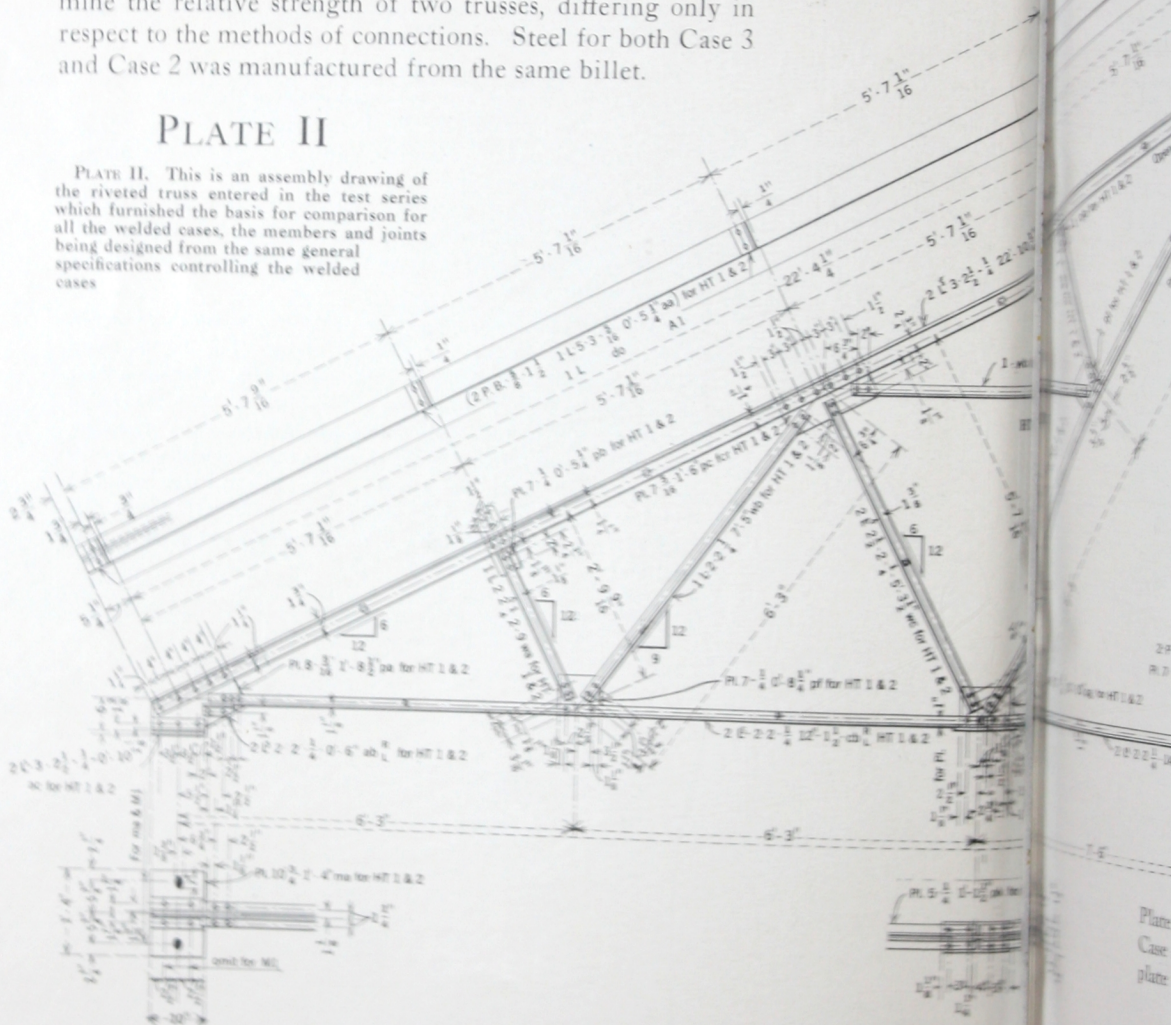
CASE 2 was the riveted truss, the layout of which is shown in Plate II. By reference to Plate I, it will be seen that the sizes of the upper and lower chords of both trusses are the same.

CASE 3

I N this case (see Plate III, page 18), the design of numbers was identical with the riveted truss Case 2, the welded design applying as an adaptation of welding to otherwise fixed conditions, the objective being to determine the relative strength of two trusses, differing only in respect to the methods of connections. Steel for both Case 3 and Case 2 was manufactured from the same billet.

PLATE II

PLATE II. This is an assembly drawing of the riveted truss entered in the test series which furnished the basis for comparison for all the welded cases, the members and joints being designed from the same general specifications controlling the welded cases



Compared with Case 1, it will be observed that improvements in the joint designs were made. As a result of the testing of Case 1, it was obvious that joints Nos. 1 and 8, while apparently adequate in design for all practical purposes, were not uniformly balanced with the strength of the remaining structure, and that all other joints were over designed. As a result of the strain gauge work on Case 1, it was apparent the arbitrary factor of

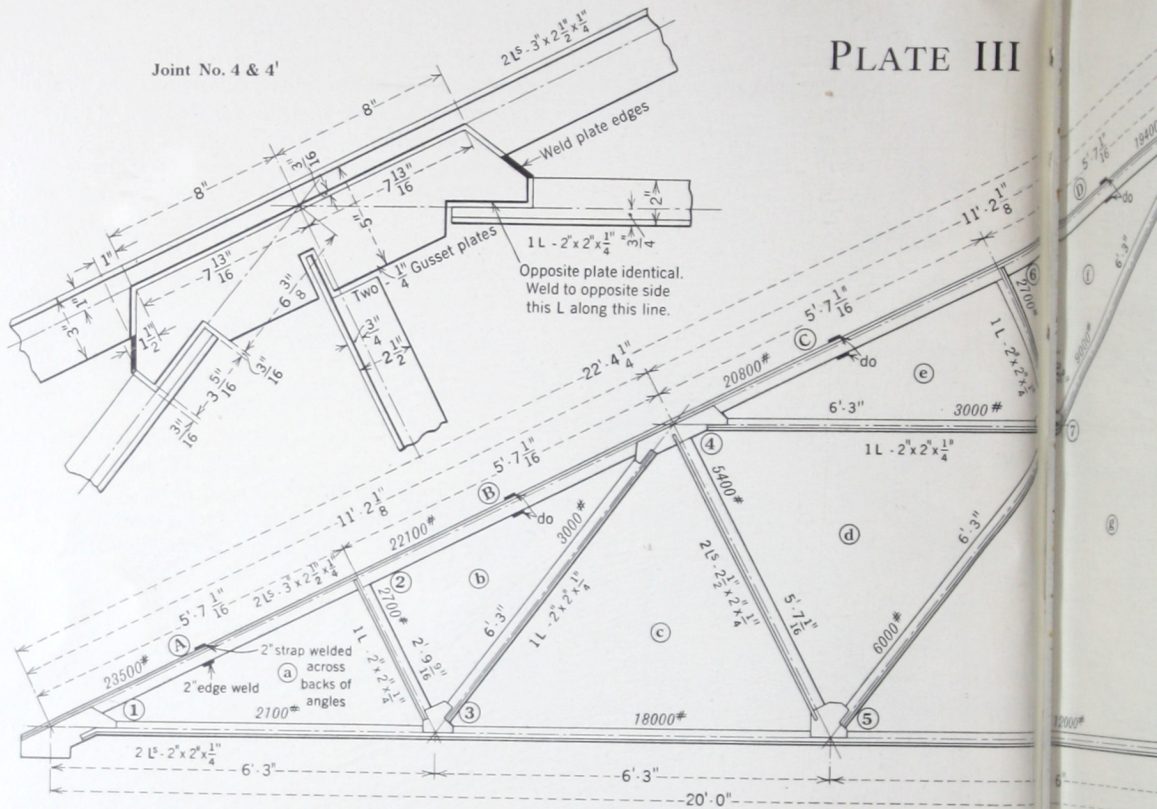
In Case 1, a rupture had occurred at point *X* (see Plate I, joint 1), due to the high bending stresses at this point. In Case 3, therefore, sufficient metal is added to reduce the bending stresses at this point so as not to exceed 16,000 lb. per sq. in. when the truss is functioning under the design load of 3000 lb. In members *a-b* and *b-c*, it is obvious that very little welding is necessary to transfer the stress into the gusset plate. The principle of the insert plate is carried out and the shear weld is figured only for transfer of stresses. However, on *b-c* an excessive weld length is provided to stiffen joint 3 against the eccentric condition caused by the single angle at this joint.

At joint 4 the gusset plate was increased in depth at the junction of the tension diagonals, and the beveled wing extending into member *c-d* was omitted, which, in addition to beveling the ends of the plates, considerably reduced the amount of welding compared with joint 4, Case 1.

At 9000 lb. panel test load, in Case 1, the gusset plate at joint 8, at Y (see

Plate I, joint 8), showed evidence of high stresses, and to relieve this condition for Case 3, the gusset plate was strengthened as shown. The horizontal extremities of the plate were shortened, thus reducing the amount of welding at this joint.

PLATE III



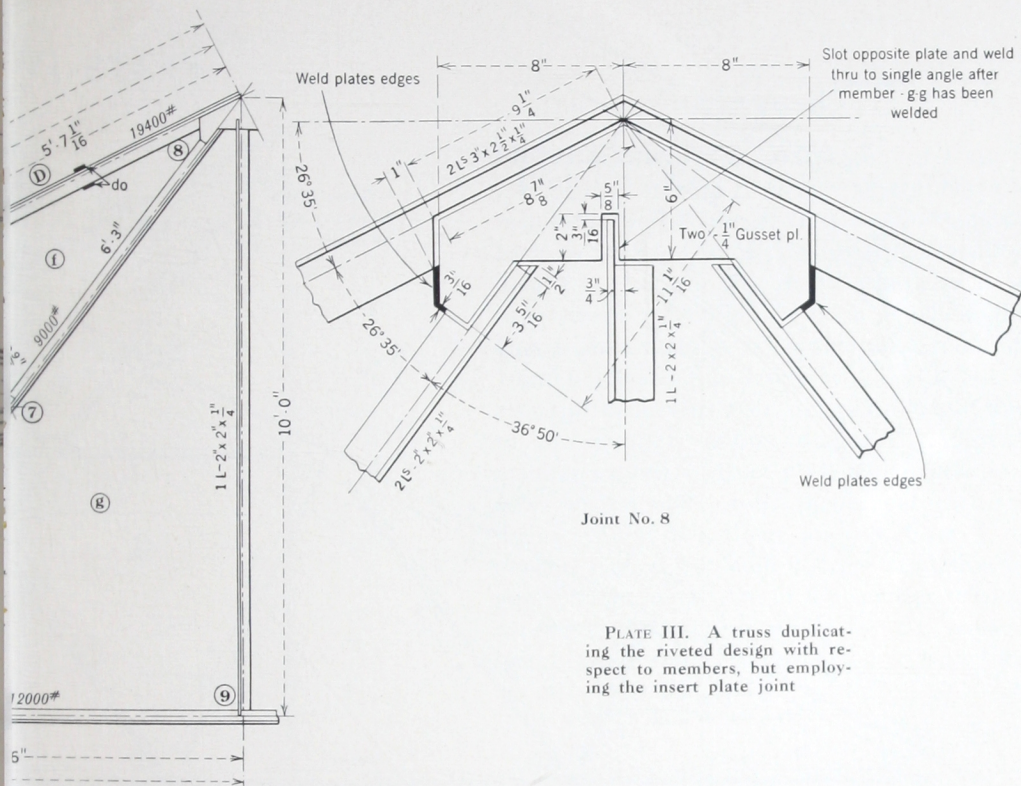


PLATE III. A truss duplicating the riveted design with respect to members, but employing the insert plate joint

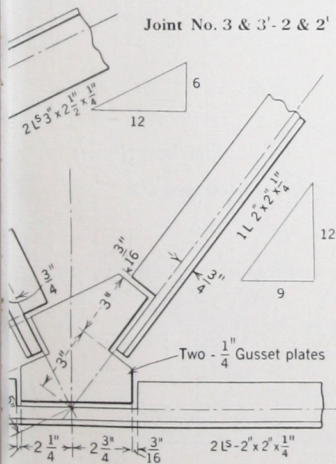
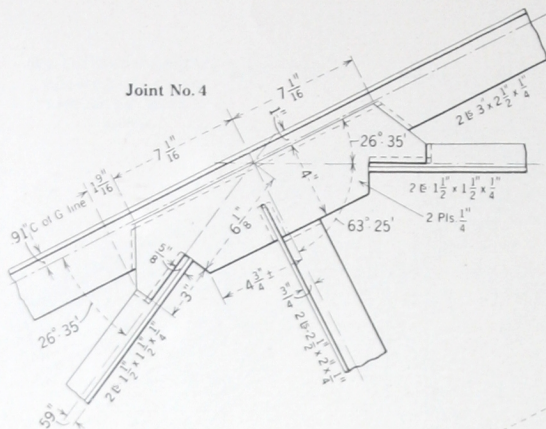
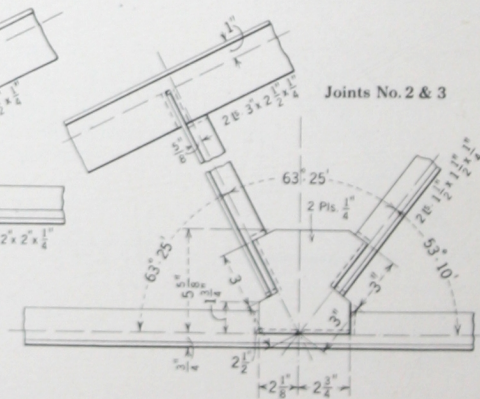
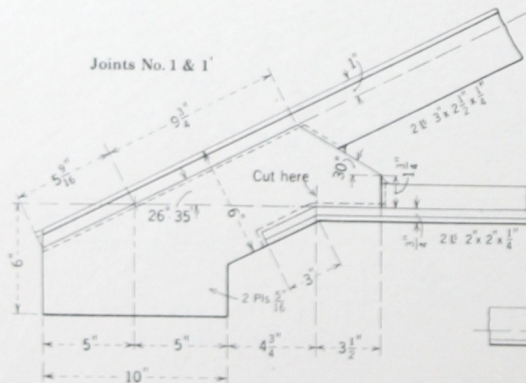
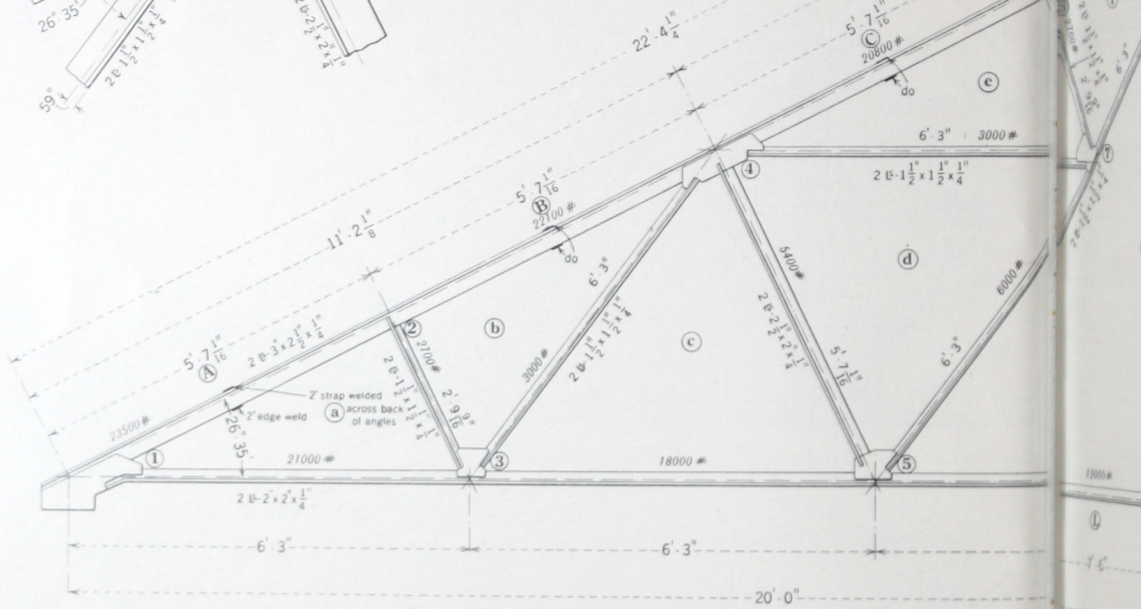
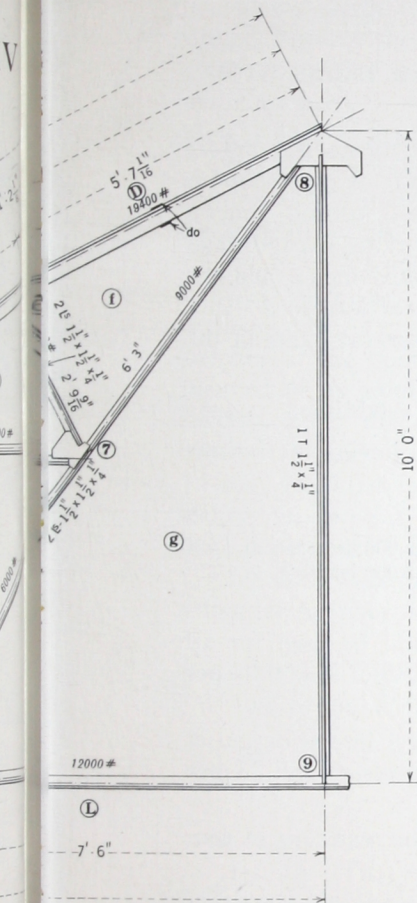


PLATE IV

Joints 1 - 1' & 8 - $\frac{5}{16}$ "

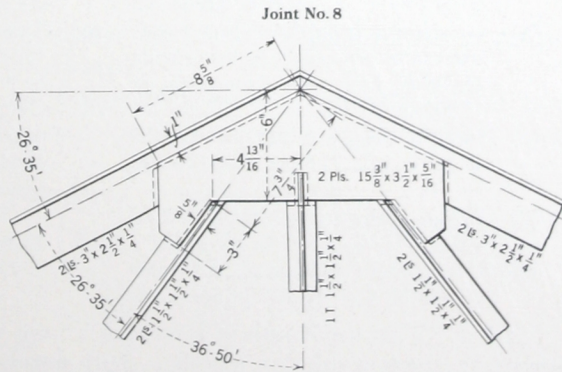
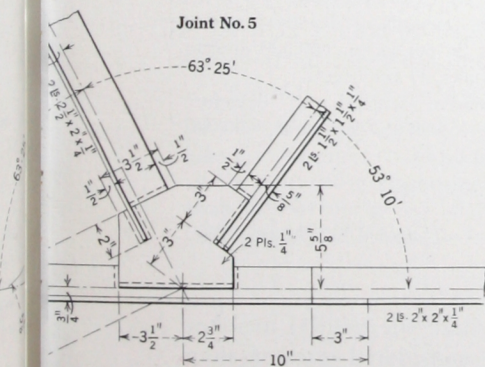
All others	$\frac{1}{4}$
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CASE 4

THIS case was a modification of Case 1, being a welded truss, having symmetrical members throughout, that is, double angles, back to back. (See Plate IV for dimension details.) Joint 3 was made symmetrical to supplement the design in Case 3, where the gusset plate metal was considerably reduced. Joint 4 follows the design in Case 3, but is accomplished with a shorter plate length due to the smaller tension diagonal. Joint 5 utilizes a smaller gusset plate which still provides excess metal over theoretical requirements. Joint 8 follows joint 8, Case 3 in general design, but with a slight reduction in gusset plate metal due to the smaller tension members. The shear welds for the tension members are reduced from $3\frac{1}{16}$ in. net to 3 in. overall.



CASE 5

SPECIFICATIONS AND DESIGN FOR A TUBULAR TRUSS WITH REINFORCED JOINTS

FOR this case, standard black pipe was used for the members. Before attempting the design of a truss, wherein the tubular or pipe shape is employed instead of the customary angles and plates, certain general assumptions had to be made with regard to practical working strengths and factors of safety, transposition of stress through deposited metal, and the utilization of plate reinforcement, that would conform or lead to a standardized engineering procedure for the design of the several joints in the tubular truss assembly.

The following design assumptions were tentatively established:

Art. 1. That open hearth pipe steel would be used, having the following average physical and chemical properties:

Carbon	.10 per cent	Ultimate Tensile	
Manganese	.40 per cent	Strength	52,000 lb. per sq. in.
Sulphur	.035 per cent	Yield Point.....	33,000 lb. per sq. in.
Phosphorus	.02 per cent		

Art. 2. That Oxweld High Test welding rod would be used for all important welds, having an average ultimate tensile strength of 58,000 lb. per sq. in. in the weld and an average yield point of 42,000 lb. per sq. in.

Art. 3. That the ultimate strength of the plate steel inserts or gusset plates equals 55,000 lb. per sq. in. in tension and compression and 75 per cent of 55,000 or 41,250 lb. in shear.

Art. 4. That the allowable working strength of the materials of construction would not exceed—

- | | |
|---|---|
| <p>(a)—Tensile strength of base metal: 52,000 divided by 4 = 13,000 lb. per sq. in.</p> <p>(b)—Compressive strength of base metal: stress to be determined by the formula $15,000-70 L/R$ with a maximum of 13,000 lb. per sq. in.</p> <p>(c)—Shearing strength of base metal: 9750 lb. per sq. in. (.75 times 52,000 divided by 4).</p> <p>(d)—Tensile strength of plates: 55,000 divided by 4 = 13,500 lb. per sq. in.</p> <p>(e)—Compressive strength of plates: 55,000 divided by 4 = 13,500 lb. per sq. in.</p> | <p>(f)—Shearing strength of plates: 10,000 lb. per sq. in. (41,250 divided by 4).</p> <p>(g)—Tensile strength of deposited metal: 13,500 lb. per sq. in. (60,000 divided by 4 times 90 per cent).</p> <p>(h)—Compressive strength of *deposited metal: 15,000 lb. per sq. in. (60,000 divided by 4).</p> <p>(i)—Shearing strength of deposited metal: 10,000 lb. per sq. in. (60,000 times 75 per cent divided by 4 times 90 per cent).</p> |
|---|---|

Art. 5. That the length of a weld at a joint where one tubular member intersects another tubular member shall not be considered greater than

the outside circumference of the least member, irrespective of the angle formed between the members joined.

Art. 6. That an insert or reinforcement plate shall be used in any joint involving stress to augment the butt weld, when the indicated diagram stress is greater than 50 per cent of the working strength of the butt weld.

Art. 7. That the combined strength of a butt weld and the shearing strength of the insert plate welds shall be not less than the allowable working strength of the respective member.

Art. 8. That where a transverse load is applied on a tubular member in excess of 50 per cent of the minimum collapsing or flattening strength of the member, a gusset or insert plate shall be employed to strengthen the member at the point of load application. The factor of 50 per cent is assumed to be ample in view of the higher collapsing strength that will develop under the conditions of load application at a joint in a tubular structure.

Art. 9. That where brazing is utilized at a joint for the purpose of sealing the interior surfaces against corrosion, the strength of such brazes shall be neglected when determining the weld design of a joint.

Art. 10. That the thickness of gusset or insert plates, when inserted into a tubular member, shall conform to the following specifications:

STANDARD PIPE			EXTRA HEAVY PIPE	
Nom. Pipe Size In.	Wall Thickness In.	Plate Thickness In.	Wall Thickness In.	Plate Thickness In.
1½	.145	¼	.200	⅜
2	.154	¼	.218	⅜
2½	.203	⅝	.276	⅝
3	.216	⅝	.300	½
3½	.226	⅝	.318	½
4	.237	⅝	.337	½
5	.258	⅝	.375	⅝
6	.280	¾	.432	⅝

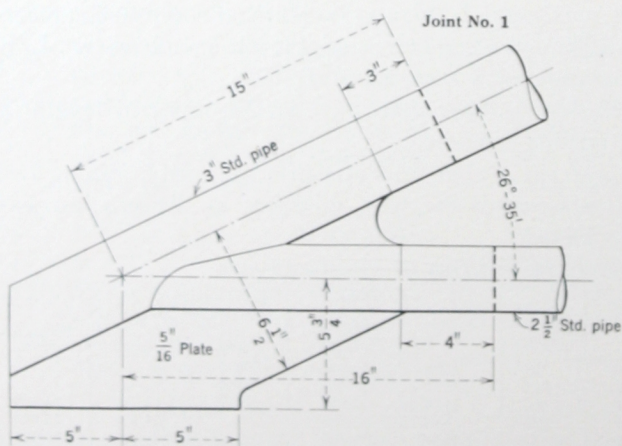
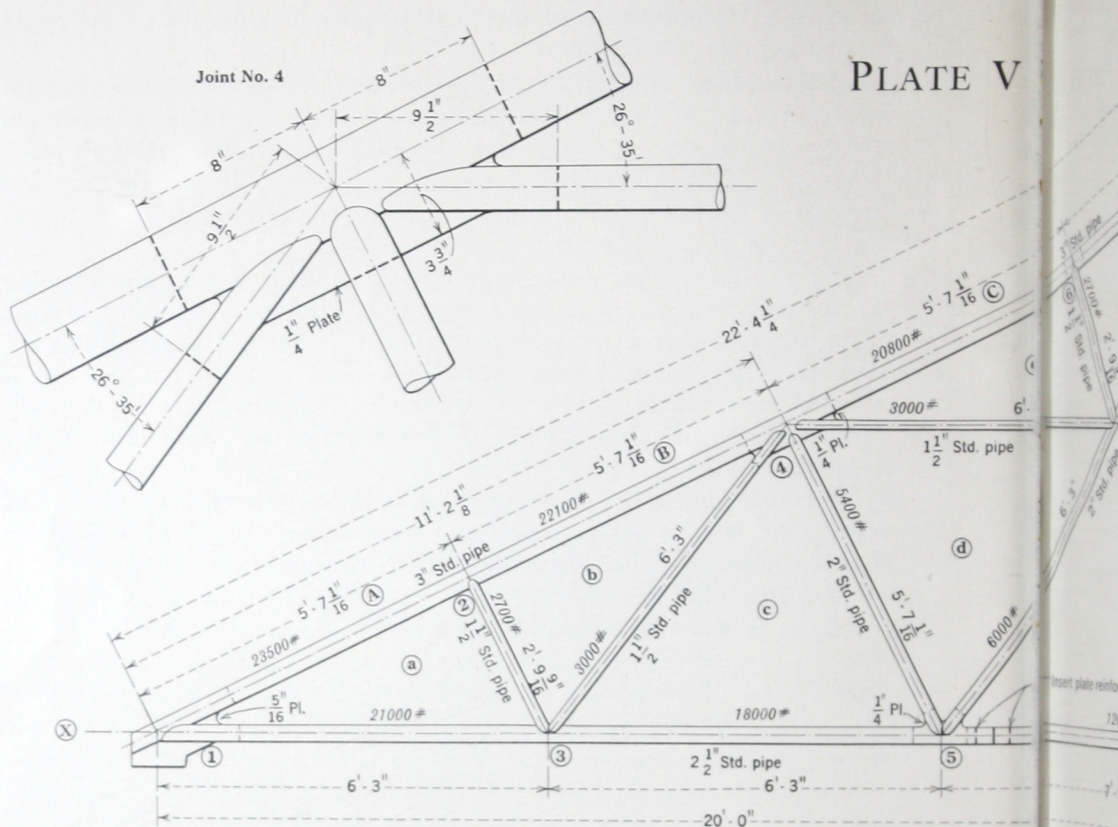
Art. 11. That the maximum L/R ratio for primary compression members shall not exceed 120; for secondary compression members, an L/R ratio of 135 shall be considered a maximum.

Art. 12. That no tension member shall have an unsupported length in excess of 50 times the outside diameter.

Art. 13. That the average design thickness of a butt weld, joining two tubular members, shall not exceed the wall thickness of the smallest size tubular member being joined.

PHYSICAL PROPERTIES OF STANDARD PIPE

Nominal Pipe Size In.	Outside Diameter In.	Wall Thickness In.	Area of Metal Sq. In.	Radius of Gyration In.	Tensile Strength of Pipe at 13,000 Lb. per Sq. In.	Tensile Strength of Insert Plates at 13,750 Lb. per Sq. In. (Slide Rule Computations)		
						¼ in.	⅝ in.	¾ in.
1½	1.900	.145	.7995	.6226	10,400	6530		
2	2.375	.154	1.075	.7871	13,950	8160		
2½	2.875	.203	1.704	.9474	22,150	9875	12,350	14,800
3	3.500	.216	2.228	1.1640	29,000		15,000	18,000
3½	4.000	.226	2.680	1.3370	34,800			20,600



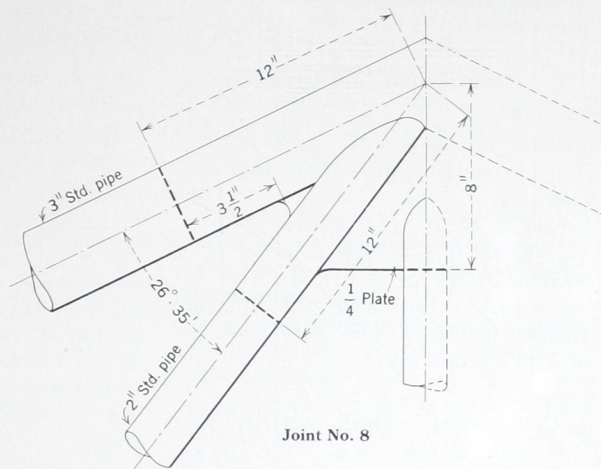
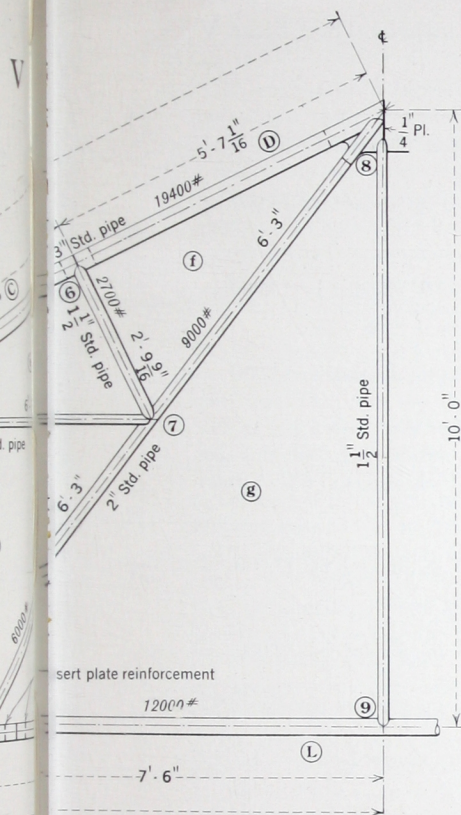
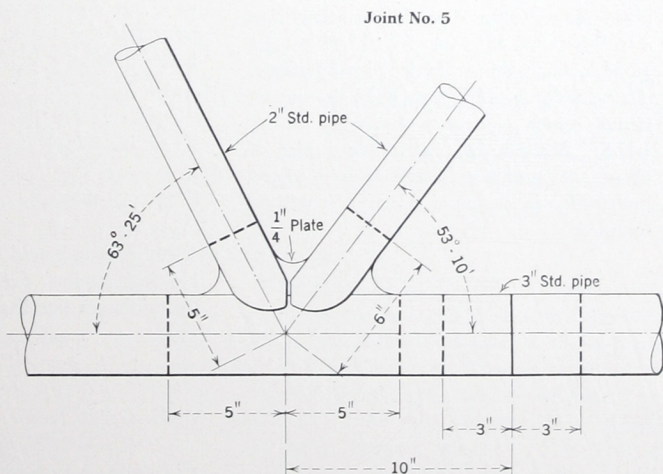


PLATE V. An adaptation of the insert plate principle to a truss constructed with tubular members. In this case the insert plate was used to reinforce the principal joints



The application of the foregoing code to Case 5 follows. See Plate V for dimension details.

DESIGN OF MEMBERS

Compression Members

Member A-a

Indicated diagram stress = 23,500 lb.

Try 3 in. standard pipe. $R = 1.1640$ (see table, page 23). $A = 2.228$.

Length = 67 in. $f = 15,000 - 70 \times 67/1.164 = 10,975$ lb. per sq. in.

$Af = 2.228 \times 10,975 = 24,450$ lb. plus or minus, which is in excess of the I.D.S. For economy, members B-b, C-c and D-d will be of 3 in. standard pipe.

Member c-d

I.D.S. = 5400 lb. Length = 67 in. Try 2 in. standard pipe. $R = .7871$. $A = 1.075$. $f_c = 15,000 - 70 \times 67/.7871 = 9036$ lb. $Af_c = 1.075 \times 9036 = 9700$ lb. plus or minus.

Similarly a $1\frac{1}{2}$ in. pipe would develop approximately 5975 lb., which is in excess of the I.D.S. While the $1\frac{1}{2}$ in. pipe would be no doubt sufficient, a 2 in. pipe will be used to insure rigid construction at this point in the truss.

Member a-b and e-f

I.D.S. = 2700 lb. Try $1\frac{1}{2}$ in. standard pipe. $R = .6226$. $A = .7995$. Length = 2 ft. $9\frac{7}{8}$ in. = $33\frac{1}{2}$ in. $f = 15,000 - 70 \times 33.5/.6226 = 11,234$ lb. plus or minus. $Af_c = .7995 \times 11,234 = 8975$ lb. plus or minus, which is greatly in excess of the I.D.S. Since a $1\frac{1}{2}$ in. standard pipe is about the minimum size that will afford preparation in a truss of this type, it will be specified.

Tension Members

Member a-L

I.D.S. = 21,000 lb. divided by 13,000 = 1.615 sq. in.

Member c-L

I.D.S. = 18,000 lb. divided by 13,000 = 1.385 sq. in.

Member g-L

I.D.S. = 12,000 lb. divided by 13,000 = .924 sq. in.

A $2\frac{1}{2}$ in. standard pipe, area 1.704 sq. in., is necessary for member a-L, and this size will be carried throughout this bottom chord.

Member d-g

I.D.S. = 6000 lb. divided by 13,000 = .46 sq. in. (plus).

Member f-g

I.D.S. = 9000 lb. divided by 12,000 = .69 sq. in.

The area of $1\frac{1}{2}$ in. standard pipe = .7995 sq. in.

The area of 2 in. standard pipe = 1.0740 sq. in.

Here again a $1\frac{1}{2}$ in. standard pipe could be used, but does not afford as good a joint preparation as a 2 in. standard pipe. The design specifies 2 in. for both members.

Member b-c

I.D.S. = 3000 lb. divided by 13,000 = .23 sq. in. (plus).

Member e-d

I.D.S. = 3000 lb. divided by 13,000 = .23 sq. in. (plus).

A $\frac{3}{4}$ in. pipe would carry the stresses, but $1\frac{1}{2}$ in. standard pipe will be used, being the minimum size that is being used in this truss.

DESIGN OF JOINTS

Joint 1

The angle formed between the upper and lower chords results in a butt weld length considerably in excess of the outside circumference of the $2\frac{1}{2}$ in. pipe; however, the outside circumference length governs. (See Art. 5 of Assumptions.) This length is 9 in. plus. In designing this joint, two assumptions can be made:

First—That the bottom chord of the truss is rigid, in which case the top chord would tend to move past the terminal of the bottom chord, causing the butt weld to function in shear.

Second—That the top chord is rigid, in which case the butt weld will function in tension.

The strength of the butt weld in the case on the basis of the design assumptions $= 9 \times .203$ (see Art. 13) $\times 10,000 = 18,270$ lb.

The strength of the butt weld in the second case $= 9 \times .203 \times 13,500 = 24,644$ lb.

As the shearing strength of the butt weld in the first case is less than the I.D.S., and as the I.D.S. is greater than 50 per cent of the working strength of the butt weld, a reinforcement plate is required. (See Art. 6.)

The required shearing strength of the insert plate for the top chord $= 23,500 \div 2 = 11,750$ lb. The shear weld length therefore equals $11,750 \div (10,000 \times .3125) = 3.74$ in. plus or minus.

Note: As the single shear value of the $\frac{5}{16}$ in. insert plate is less than the double shear value of the pipe wall the former governs. $(.3125 \times 10,000 = 3125$ lb. $2 \times .216 \times 9750 = 4200$ lb.) See Art. 4 (c) and (f).

The design provides greatly in excess of 3.74 in. length and could be modified.

The required shearing strength of the insert plate for the bottom chord $= 21,000 \div 2 = 10,500$ lb. The shear weld length therefore equals $10,500 \div (10,000 \times .3125) = 3.36$ in. The design provides $2 \times 3 = 6$ in. The minimum tensile strength of the insert plate in member $a-L = 2.875 \times .3125 \times 13,750 = 12,350$ lb., which exceeds the required shearing strength of 10,500 lb.

From the above analysis, it is evident that

the design of this joint provides for sufficient deposited metal to develop a strength appreciably in excess of the stresses that will be imposed at this point.

The joints between the insert shear weld and the butt welds may be sealed against weather by bronze fillet welds.

Joint 2

There are two factors at this joint to be considered, viz., the compressive strength of the butt weld, and the collapsing strength of the upper chord at this point.

The compressive strength of the weld $= \pi \times 1.90 \times .145 \times 15,000 = 13,000$ lb.

The collapsing strength of a 3 in. standard pipe is approximately 10,000 lb. As the I.D.S. and the transverse load at this joint are less than 50 per cent respectively of the above strengths, no reinforcement plate is required.

Joint 3

Member $a-b$ I.D.S. $= 2700$ lb. in compression. The compressive strength of the butt weld equals 13,000 lb. (see Joint 2). The collapsing strength of a $2\frac{1}{2}$ in. standard pipe is approximately 8000 lb. Therefore no reinforcement plate is required.

Member $b-c$ I.D.S. $= 3000$ lb. in tension. The tensile strength of the butt weld $= \pi \times 1.90 \times .145 \times 13,500 = 11,700$ lb. Here again a reinforcement plate is not required.

Joint 4

Member $b-c$ I.D.S. $= 3000$ lb. in tension. The tensile strength of the butt weld as shown above $= 11,700$ lb. No plate is required.

Member $c-d$. The same holds as for joint 4.

Member $c-d$ I.D.S. $= 5400$ lb. in compression. The compressive strength of the butt weld $= \pi \times 2.375 \times .154 \times 15,000 = 17,250$ lb.

The transverse load on the upper chord tending to collapse the member equals the algebraic sum of the parallel components of the forces acting in web members. The components of the forces in members $b-c$ and $d-e = 3000 \times \sin 26^\circ 35' = 1342$ (plus or minus) $\times 2 = 2684$ lb. acting towards joint 5. The force of 5400 lb. in member $c-d$ acts in the opposite

direction; therefore the resultant force which is the transverse load = 2716 lb. The collapsing strength of upper chord, as before, is approximately 10,000 lb.

The above figures of strength do not indicate that a plate is necessary. However, a plate is provided at this joint for the purpose of stiffening the truss at the center panel.

Joint 5

Member *c-d* I.D.S. = 5400 lb. in compression. The compressive strength of the butt weld = 17,250 lb. (see joint 4). No plate necessary.

Member *d-g* I.D.S. = 6000 lb. in tension. The tensile strength of the butt weld = $\pi \times 2.375 \times .154 \times 13,000 = 15,500$ lb. plus or minus. Here again the I.D.S. is less than 50 per cent of the weld strength, requiring no reinforcement. The horizontal components of the forces acting in members *c-d* and *d-g* towards joint 9 impose a compressive or crumpling effect upon the upper metal in the lower chord, immediately behind the joint, in the *g* panel of the truss. Assume that this effect is confined to approximately one-fourth the circumference, or $\pi \times 2.875 \div 4 = 2.26$ in. The compressive strength of this portion of the member resisting these forces = $2.26 \times .203 \times 13,000$ lb. (see Art. 4-b) = 5960 lb. plus or minus. These forces = $5400 \times \sin 26^\circ 35'$ plus $6000 \times \sin 36^\circ 50' = 6000$ lb. plus or minus. While the forces are closely in balance, it is desirable to reinforce this joint, it being one of the principal points of stress in the diagram. The members selected at this joint restrict the plate thickness to $\frac{1}{4}$ in. (see No. 10). It can be assumed, no doubt, that the insert plate will carry the horizontal force of 6000 lb into the bottom chord through the shear welds. As the single shear value of the plate governs, the shearing strength per inch of weld = $.25 \times 10,000 = 2500$ lb. Therefore the necessary weld length = $6000 \div 2500 = 2.4$ in. The design provides considerable in excess of 2.4 in. and provides stiffening of the web members entering this joint.

Joint 6

This is the same as joint 2.

Joint 7

This is the same as joint 3.

Joint 8

Member *D-f* I.D.S. = 19,400 lb. in compression.

Member *f-g* I.D.S. = 9000 lb. in tension.

In order to obtain the direct stresses acting on the ridge weld, the force of 19,400 lb. has been resolved into its horizontal and vertical components, which act in direct compression and shear respectively.

Vertical component = $19,400 \times \sin 26^\circ 35' = 8680$ lb.

Horizontal component = $19,400 \times \cos 26^\circ 35' = 17,350$ lb.

Required shearing area = $8680/10,000 = .868$ sq. in.

Required compression area = $17,350/15,000 = 1.155$ sq. in. or a total sum.

The weld area, governed by Art. 5, is approximately $\pi \times 3.5 \times .216 = 2.38$ sq. in. plus or minus, indicating sufficient strength on the above design assumptions.

The weld strength of member *f-g* = 13,500 lb. (see joint 5), 50 per cent of which is less than the I.D.S. of 9000 lb. Therefore a reinforcement plate is required. A $\frac{1}{4}$ in. plate has been selected.

The unit shearing strength of the plate weld = $.25 \times 10,000 = 2500$ lb. The stress to be carried by the plate = $9000/2 = 4500$ lb. Therefore the shear weld length = $4500/2500 = 1.8$ in. The design provides approximately 3 in. Likewise the shear weld length for the upper chord insert = $19,400/2 \div 2500 = 3.88$ in. This length is greatly exceeded by the necessary dimensions of the plate. As member *g-x* carries no static stress, the design provides for a fillet contour weld where the slotted metal engages the plate, the member inserting the plate a sufficient distance to insure ample strength.

Joint 9

Member *g-x* I.D.S. = 0. Design provides a direct butt weld that should develop the strength of the base metal of member *g-x*. However, an insert plate has been provided to insure ample strength. The design provides 4 in. each side of the center line of the truss.

CASE 6

MEMBER DESIGN

The design procedure for this case differed from that governing Cases 1, 3 and 4, in that the principal tension members were designed with respect to the factor of safety of the upper chord, which necessitated the adoption of maximum tensile strength factors for members functioning in tension, the objective for this case being a closer approach to a *minimum distribution of metal with respect to the maximum factor of safety of the truss*.

The factor of safety governing the design of members is obtained by investigating the strength of a panel length of the upper chord, the slenderness ratio of which equals 70.5. The probable maximum unit load for a column of this type having a fixation factor of 1.5 and a slenderness ratio of 70.5 is 34,000 to 35,000 lb. per sq. in. (see Bulletin No. 218, Bureau of Standards, pp 656-7 and 662-3), or approximately the yield point of mild steel. Due to inaccuracies in the test structure and from experience obtained in the initial truss tests, the above unit load was reduced 15 per cent.

For the truss members functioning in tension, a unit strength of 36,000 lb. per sq. in. was used for the average yield point of the material, it being assumed that, for practical stability of the structure, the yield point of the material in the tension members should be approached at a point when the upper chord member failed. This assumption, no doubt, penalizes the tension members with weight and would be too conservative for practical construction, but it affords opportunity to study column and joint action in this case under very stable web conditions.

On the above assumptions, the detail designs of a few typical members follow (see Plate VI for dimension details):

Upper Chord

2 angles, 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ in., long legs vertical, greatest I.D.S. at normal load, 23,500 lb.; area, 2.64 sq. in.; R , 1.00 and .95; diagram length, 67. Slenderness ratio $67 \div .95 = 70.5$. Probable maximum unit load equals

85 per cent of 35,000 lb. or 29,750 lb. Maximum column strength equals $29,750 \times 2.64 = 78,540$ lb. Design factor of safety = $78,540/23,500 = 3.34$ plus or minus.

Strut c-d (Compression)

I.D.S., 5400 lb.; maximum stress at failure, 5400×3.34 or 18,036 lb.; diagram length, 67 in. In the trusses, Cases 2, 3 and 4, this strut was made up of two angles, $2\frac{1}{2}$ by 2 by $\frac{1}{4}$ in., area 2.12 sq. in., having a minimum slenderness ratio, back to back, of $67 \div .78 = 86$, indicating a maximum unit load of approximately 34,000 lb. per sq. in. plus. Therefore, the maximum column strength equals $34,000 \times .85 \times 2.12 = 61,250$ lb. = 3.3 times the maximum indicated stress.

Try 2 angles 2 by 2 by $\frac{1}{4}$ in. back to back. Area equals 1.88 sq. in. Least r equals 0.61. Slenderness ratio equals 110, indicating a maximum unit load of 33,000 lb. per sq. in. Therefore the maximum column strength equals 33,000 or 85 per cent $\times 1.88 = 52,750$ lb., which is 2.93 times the maximum stress.

While it appears theoretically that material reduction in area of metal in this strut can be made compared to the original selection of two $2\frac{1}{2}$ by 2 by $\frac{1}{4}$ in. angles, caution is necessary to provide against secondary bending stresses likely to occur in the light shapes under load, which, added to the direct compressive stresses, might sum up sufficient stresses to cause failure materially below the normal strength of the strut. For the present it was decided to use 2 by 2 by $\frac{1}{4}$ in. angles back to back.

The design of compression members *a-b* and *e-f* followed similar procedure. Tee sections were used of a strength considerably over theoretical requirements. For tension members the design of lower chord *a-l* can be taken as typical. I.D.S. = 21,000 lb.; maximum stress at failure = $21,000 \times 3.34 = 70,140$ lb.; required area, $70,140/36,000 = 2$ sq. in. plus or minus. A 3 by 3 by $\frac{5}{16}$ in. stock tee has an area of 1.95 and answers the requirements for both strength and design.



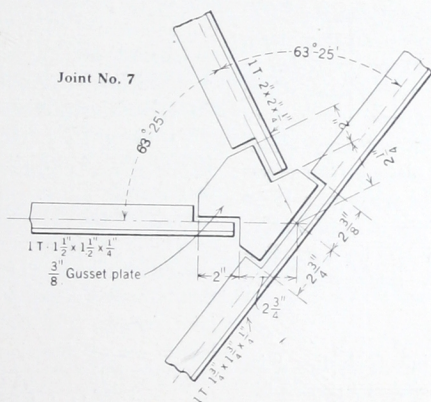
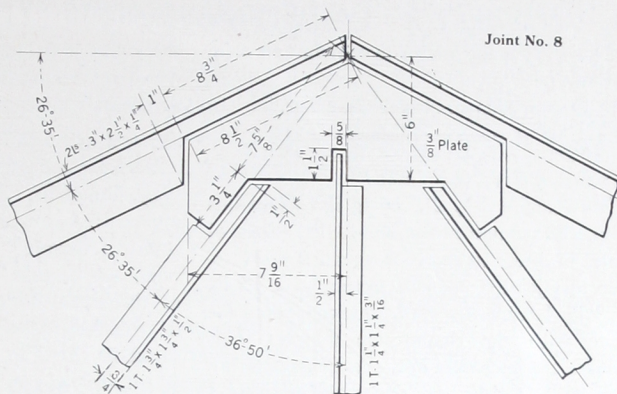
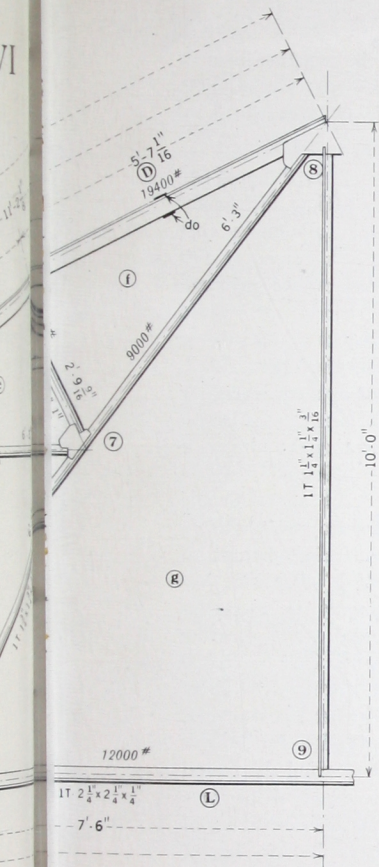
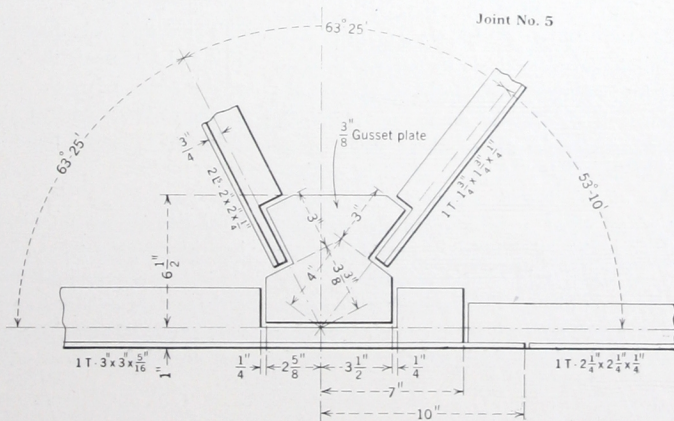


PLATE VI. For this case an attempt was made to develop the minimum weight a truss might attain and still retain a strength equal to the riveted design of Case 2. Therefore, the design represents a closer approach to a theoretical disposition of truss materials than any previous case



DESIGN OF JOINTS

Design assumptions for maximum unit weld stresses:

Compression . . .	40,000 lb. per sq. in.
Tension	36,000 lb. per sq. in.
Shear	36,000 lb. per sq. in.

Member a-A (Compression)

Two angles 3 by $2\frac{1}{2}$ by $\frac{1}{4}$ in. Gage line 1 in. Maximum stress 79,540 lb. Area of metal in direct compression equals 2.64 sq. in. For this joint a one-half section of a 15 in. 42.9 lb. I-beam was used, thus combining the plate with the compression flange. This avoided the long weld in the compression member, characteristic of the previous welded trusses. Although the metal area opposing member *a-A* is sufficient to develop at 40,000 lb. per sq. in., the maximum diagram load, an insert shear weld of 2 in. was provided to insure rigidity at the connecting point. The selection of a 15 in. I-beam was necessary to provide sufficient web depth and flange width to accommodate the joint design. An excess of metal resulted, due to the profile of a 15 in. 42.9 lb. I-beam, but this was not sufficient to materially affect the economy gained by eliminating the long double-V shear weld mentioned above.

Member a-L (Tension)

One tee 3 by 3 by $\frac{5}{8}$ in. Gage line 1 in. Area = 1.95 sq. in. Maximum stress 70,140 lb. Strength of tension weld = $2 \times .3125 \times 36,000 = 22,500$ lb. Balance for shear weld = $70,140 - 22,500 = 47,640$ in. As the minimum plate thickness governs the unit shearing strength, each inch of shear weld develops $1 \times .3125 \times 36,000 = 11,250$ lb. Therefore the length of shear weld equals $47,640 \text{ lb.} \div 11,250 \text{ lb.} = 4.23$ in. Three inches of this length is provided for by the horizontal shear weld next to the member, and an additional 3 inches is provided by the inclined shear weld coinciding with the bevel of the gusset plate. This latter detail not only insures ample strength for the tension member joint, but develops a high resisting moment in

the gusset plate, against the negative moment developed at the same point by the reaction, or one-half the total dead and test loads.

Joints 2 and 6 (Compression)

Member a-b. One tee 2 by 2 by $\frac{1}{4}$ in. Gage line $\frac{3}{4}$ in. Area = 1.05 sq. in. Maximum stress = 9018 lb. Assuming that sufficient deposited metal was placed at this joint to develop the compressive strength of the strut, failure in such a case would be expected either by the crushing of the metal in the 3 in. legs of the upper chord, just inside the gage line, or by a straight shear through the upper chord adjacent to the lateral legs of the tee strut. Checking the former, it can be assumed that the crushing effect would be applied over a length of approximately 2 in. minimum. As there are two angles in the upper chord, the compression strength of this section = $2 (2 \times .25 \times 40,000 \text{ lb.}) = 40,000$ lb., which is greatly in excess of the maximum stress imposed. The shearing strength of the upper chord is $2.64 \times 36,000 = 95,000$ lb., 60 per cent of which is in direct shear with the plane of the strut. It is obvious that excessive metal is provided for by this joint design. The welds securing the lateral legs of the tee function in double shear for a length of 2 in. In addition the normal leg of the tee is welded directly to the 3 in. legs of the upper chord.

The design procedure given above for joints 1, 2 and 6 presents the more unusual features of this truss and at the same time is indicative of the methods used at the remaining joints. Adjacent to joint 5 there is a field weld in the lower chord. The maximum stress in this member, a $2\frac{1}{2}$ by $2\frac{1}{2}$ by $\frac{1}{4}$ in. tee = 40,080. Since the unit tensile strength used for designing the member coincides approximately with the unit tensile strength for designing the weld, a shear weld is unnecessary. However, to insure ample strength, a 3 in. shear weld length was provided.

Fabrication of Welded Trusses

THE available facilities for conducting the experimental work involved in the development of these cases confined the testing structure and truss fabrication to a space approximately 13 ft. wide by 50 ft. long. This precluded the possibility of working out efficient shop methods. However, progress was made with each case as developed, giving sufficient experience to permit the promulgation of a Procedure Control for the first commercial application.

The oxygen consumption for each welded truss was measured by recording the gauge pressure before and after each detail tacking and welding operation. These various pressures were referred to a pressure temperature volume chart for volume of oxygen consumed. A thermometer was secured to the side of the oxygen cylinder to record temperatures at intervals during consumption periods. The acetylene consumption was calculated from the oxygen consumption by using the proportional ratios for cutting and welding, applying to the respective thicknesses of metal involved. The items of acetylene consumption appearing in the cost supplements should be correct to within 5 per cent. Deposited metal was accounted for by weighing the rods consumed by each tacking and welding operation.

All cutting was done with a No. 1 (Oxweld) cutting nozzle. Welding was done with No. 10 and No. 12 (Oxweld) welding tips. Time studies of typical joints are given in the appendix and show a welding time relationship between these two sizes of tips for the same type of joint.

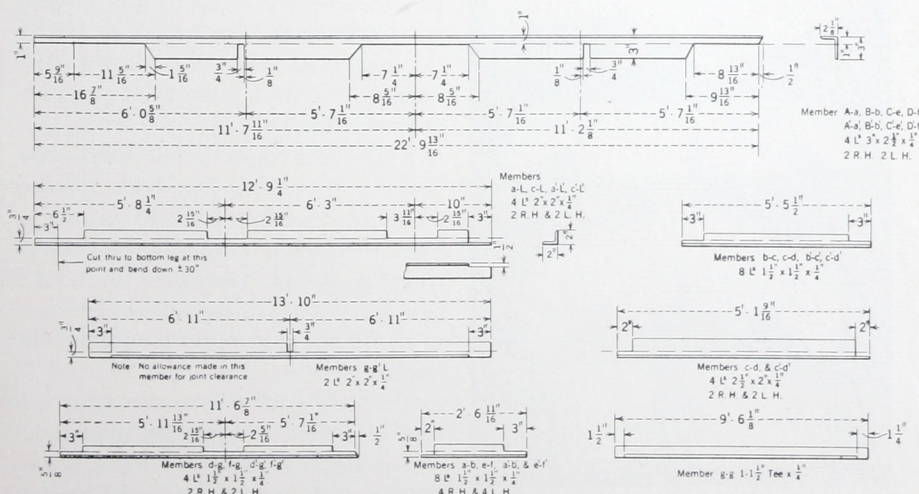


FIG. 4. Typical member templates for a welded truss composed of structural steel members

WELDED FABRICATION PROCEDURE

Before the actual shop work on the first experimental truss was started, the welders selected to do the welding were required to qualify by welding together two 9 by 18 in. plates, of a thickness and with a joint preparation resembling that to be ultimately worked upon. The square plate was cut into coupons 2 in. wide by 18 in. long, and tested in a tensile machine. The strength requirement for the welds for structural steel were set at a minimum of 50,000 lb. per sq. in., and to withstand bending through 180 deg. These requirements were met forthwith by the men selected. The results of the tensile test on these coupons showed a strength of 58,000 to 60,000 lb. per sq. in.

As in the design development, so in the fabrication procedure, experience gained on each truss was used to perfect methods on the subsequent trusses. By the time Case 4 was ready for fabrication, a somewhat regular procedure had been developed. This will be described in detail. A space of $\frac{1}{4}$ in. was allowed for all joint spacing. On the other hand the cuts were not beveled for welding as is often done for metal $\frac{1}{4}$ in. thick or more.

Layout and Preparation:

Heavy paper templates, which Fig. 5 typically illustrates, were made for the necessary cutting of all gusset plates and members. The templates for the gusset plates were transferred to a $\frac{5}{16}$ in. stock plate. The profiles

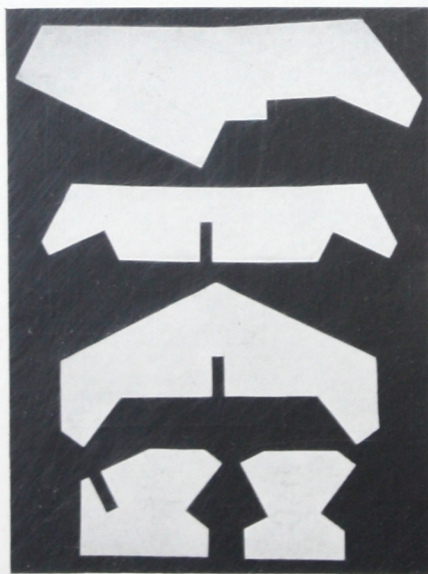


FIG. 5. Typical insert plate templates for a truss composed of structural steel members

were made with soapstone pencils, and at each change of direction of profile lines, a visible center punch mark was made. The cutter, using light brown goggles, had no difficulty in following these lines. For the long profile lines a steel flat straight-edge was used to guide the blowpipe, two straight-edges being used, one cooling in water. The edges of the templates were made to coincide with the edges of the next adjacent template outline whenever this could be done. Thus, a single blowpipe cut, in many cases, isolated two template edges.

In laying out the members, a steel tape was used to establish the insert center lines. The two angles composing each inserted member were clamped together, and the inserts for

one side were laid out. Inserts for the opposite member were then transferred from the first side by sighting over a small square. In cutting the inserts, multiple operation was tried, but it was found more expedient to cut one angle at a time. It was found also that time could be saved at no expense to workmanship on the insert plates by considerable free hand cutting.

Preliminary Assembly:

The principal members were laid back to back with the vertical legs standing up, and C-clamped together. The double gusset plates for each joint were next inserted and tacked at two places. The next operation consisted in welding the gusset plates into their respective members. Each member was laid across blocking, spaced from 6 to 8 ft. apart, with the plane of the gusset plates in a horizontal position, so that horizontal welding could be done. The joint along the gage line was welded first, then the leg welds—either side. The member was then turned over and welded on the opposite side. At the completion of this operation, the assembled members were straightened with the aid of a heating blowpipe (a welding blowpipe with a No. 12 [Oxweld] tip) by springing the member on its back at the joint. Little difficulty was experienced here as the deflection was not over $\frac{1}{2}$ to $\frac{3}{4}$ in. end to end. The following advantages are gained by this method of preliminary assembly:

1. The major part of the welding can be done most advantageously.
2. The partially welded members can readily be straightened.
3. The principal or longest welds are isolated from the final welding operation, which reduces the weld contraction effect in the truss as a whole—in fact, to a point where straightening of the truss is unnecessary.
4. Heating is conserved, as the reverse side weld can be made as a continuation of the first weld by turning the member over. It is here that a special fixture for assembling, holding and turning the member would provide better efficiency than was possible under the conditions at Buffalo.

Final Assembly:

The partially welded principal members and the compression strut of a half truss section were next laid horizontally on stringers and C-clamped in their proper position. These consisted of members: upper chord (*A-a, B-b, C-c, D-d*), lower chord (*a-L, c-L*), tension diagonal (*d-g* and *f-g*), and compression strut (*c-d*). Heavy cardboard templates cut with an angle of 26 deg. 35 min. were used to square the assembled members. The remaining web members were next placed and clamped, and welding was resumed. The welding procedure was consecutive, starting at joint 1 by welder No. 1, and at joint 8 by welder No. 2, and meeting in the center. Each welder was permitted to work on the joint most handy to himself. The welding application at a typical joint is shown in Fig. 6. At completion the truss

was in good alignment in both planes and required but little straightening.

In Case 6, because of the use of tee members below the top chord, the layout was further simplified. A slight modification was made in the preliminary assembly of this

case, consisting in the omission of one of the gusset leg welds. In other words, only the gage line weld and one leg weld of the gusset plate were made, the other leg being left for the final assembly. This procedure reduced the straightening time at the completion of the preliminary assembly. It also facilitated the work of aligning the half sections in the final assembly. It would probably be better to omit both leg welds in the preliminary

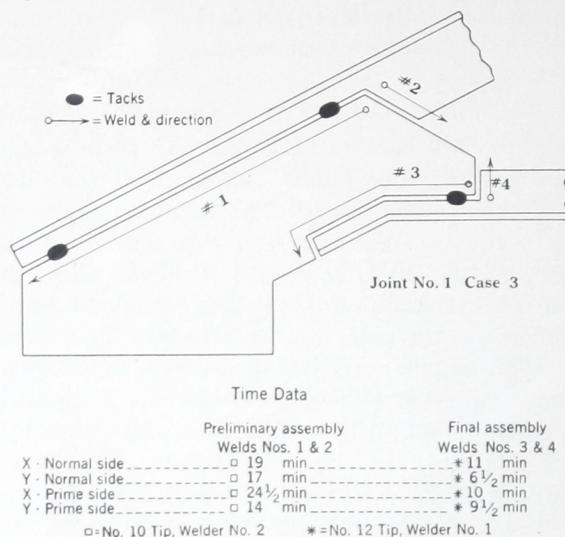


FIG. 6. Diagrammatic representation of welding procedure used for this type of joint

assembly. Experience with the final assembly brought out the possibilities for further economies in this type of construction by using tee section members.

Case 5, the pipe truss, has been left for separate and detailed consideration because of the importance of the special methods developed.

Layout and Preparation, Case 5:

In the shop development of this truss, an attempt was made to originate a simple practical method of procedure for the assembly operation. (The nature of the member material introduces complications for the layout man in regard to centering the joints, unless a different method is employed than is usually followed where tubular materials are substituted for structural shapes.)

Two sets of templates were prepared—one set providing the terminal or profile cuts, the second set establishing the joint or template base line. The latter is combined with the member templates for insert slots and enables the layout man to indicate either or both details at each joint. These base lines are shown dotted on the member templates and, when transferred to and center punched on the member, readily provide a means of setting up the truss web member. Further, these base lines guide the layout man

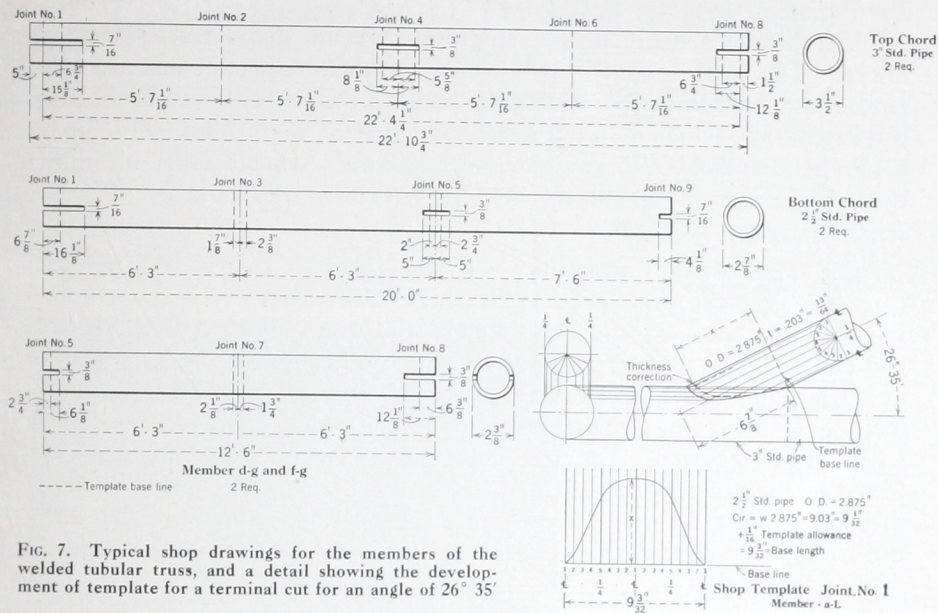


Fig. 7. Typical shop drawings for the members of the welded tubular truss, and a detail showing the development of template for a terminal cut for an angle of $26^{\circ} 35'$

when transferring the terminal or profile templates; they appear also on these latter templates.

Experience showed the practicability of this method. Typical details of the two sets of templates for this case are shown in Fig. 6. The layout of the members consumed considerably more time than in other cases, primarily due to the experimental conditions. Greater caution was exercised in doing this work which also is responsible for the longer layout period. For a practical case, where there would be a multiplicity of trusses, it would no doubt be better to divide the layout work, using one operator for base lines and slots, and the other for terminal profiles.

As designed for this case, the insert plate templates require more free hand cutting than in the previous types. However, the time was reduced to approximately one half compared with Cases 3 and 4.

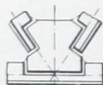
Considerably more cutting is required for the tubular truss than for the shape trusses, and the nature of the cuts slows up the operator. The members must be rotated during the cutting operation for all profile cuts, and reversed for the insert slot cuts. For production work of this class, properly designed fixtures would greatly facilitate the operator's work and lower the cost of preparation. By striking the cuts with a ball-pen hammer, the adhering oxides are readily dislodged.

Preliminary Assembly, Case 5:

A similar procedure to that employed for the shape trusses was followed. The insert plates were inserted into their respective member slots, tacked and welded. The plate fillet welds were not made until the truss was assembled, which resulted in the omission of these fillet welds under a tubular member. No straightening of members was necessary at completion of welding and no special welding procedure was followed.

Final Welding, Case 5:

The final assembly procedure was also similar to that used in previous cases, namely—the principal members were laid horizontally on a supporting scaffold, lined up and tacked; then the web members were filled in and tacked. Flat wedges were used to adjust the height of each member at a joint, and where gusset plates occurred, the wedges were inserted in the slots which jammed the member at the insert plate. After tacking, the wedges were removed. The profile welds were made first at each joint followed by the exposed fillet plate welds. Each half truss section was rotated to accommodate the welders. From the experience gained in fabricating this truss, it was obvious that the joint designs could be modified with respect to the insert detail.



Testing Structure and Methods

TEST STRUCTURE

IN the development of a method for testing the six trusses involved in this experimental program, an attempt was made to dispose the applied loads in conformity with the theoretical load increments which govern the calculations for determining member stresses. The theoretical investigation contemplated reconciliation of the theoretical stresses with the actual stresses in the respective truss members under various applied loads; also, the question of residual weld stress effect, on such type of structures, was involved.

In order to render this theoretical work more intelligible, and with as few factors of assumption as possible, it was decided to apply the test load at each truss panel point, concentrated in such a manner as to cause the center of gravity of each panel to pass through the center of gravity intersection point of the members at each joint.



FIG. 8 (*above*). View of tubular truss showing the amount of pig iron sustained up to the point of failure

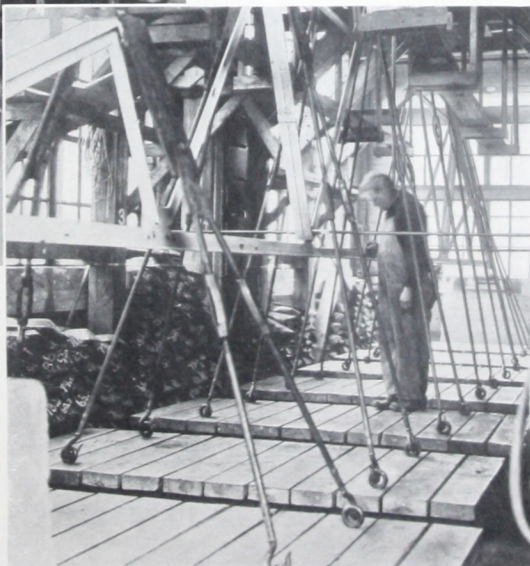
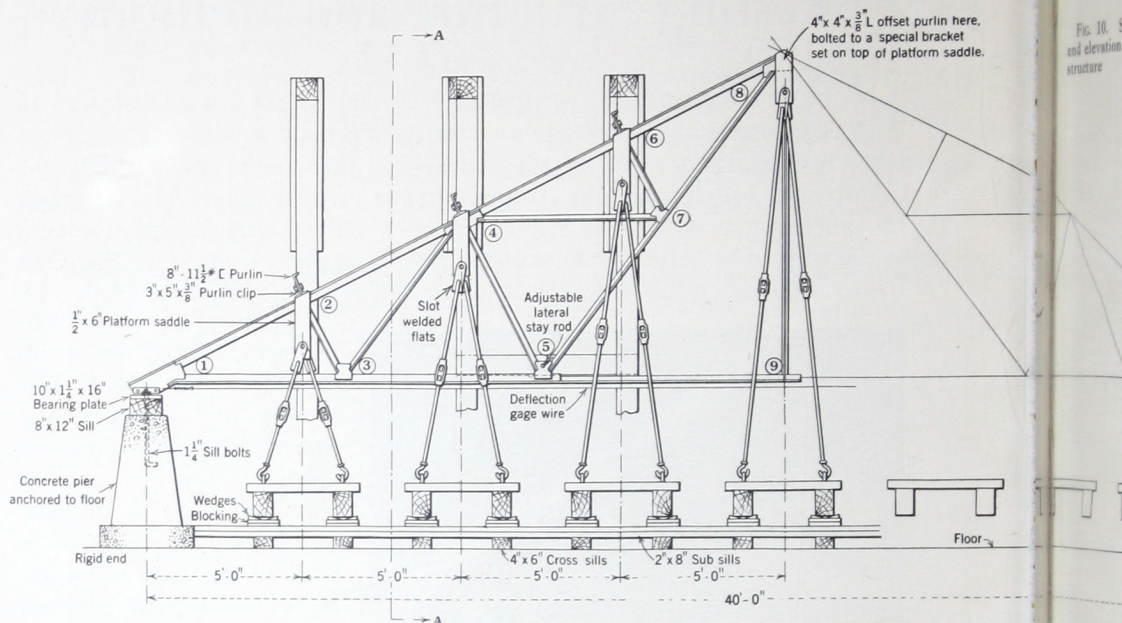


FIG. 9 (right). Typical view of the testing structure showing load platform suspended from each panel point



Heretofore, as far as can be determined, similar structures were tested by applying a superimposed load, generally made up of bags of sand, over roof planking, laid across the roof purlins. The factors of error known to exist as a result of such methods of testing, such as arch action, uneven load distribution, intermittent increments of live load and impracticability of testing to destruction, were thereby avoided. The drawing and photographs, Figs. 8, 9 and 10, show the test structure in detail.*

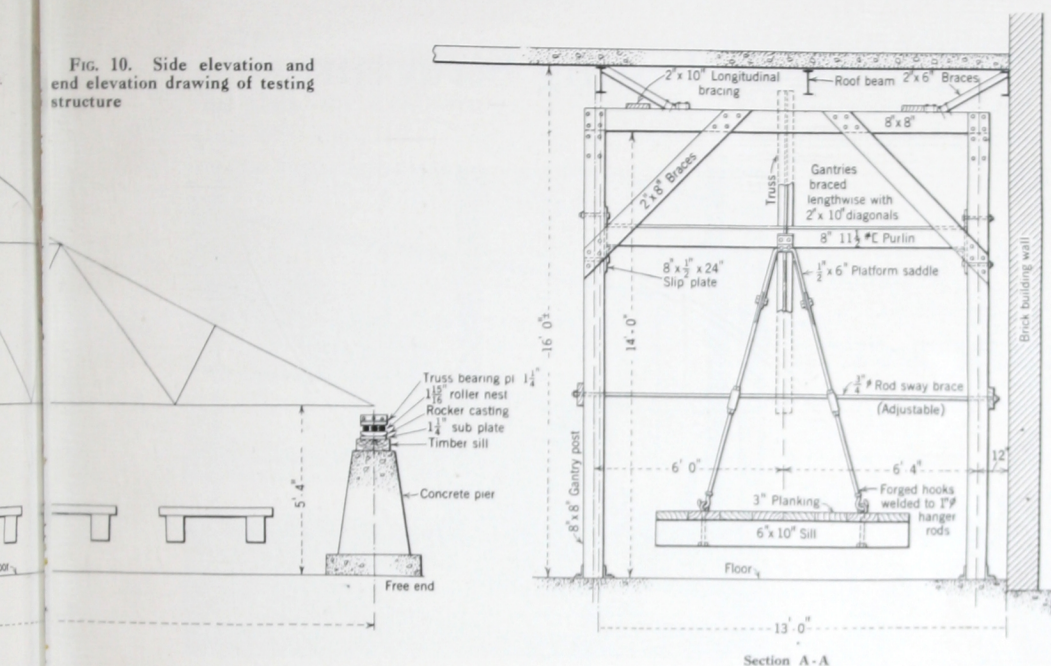
For loading material, pig iron was used, each pig being accurately weighed and marked to one-hundredth of a pound.

A deflection gauge wire was stretched under the lower chord and secured to the gusset plate of joints 1 and 1' by a straddle wire over a 1/4 in. bolt passing through the plate close to the center line of reaction. A heavy coil spring and turn-buckle served to draw the gauge wire taut. Foot planks and portable scaffolds were provided throughout the test structure to facilitate the strain gauge work, purlin leveling, painting, hammer testing and observations. Removable blocking and wedges were provided beneath the suspended platforms to restrict the drop of the suspended platforms when approaching maximum truss strength.

The assembled truss was elevated to position on the concrete supporting piers, and by reference to a true line stretched over the gantry beams, align-

*The writer makes acknowledgment to Structural Department of the Bureau of Standards for many valuable suggestions made in connection with the development of the testing structure.

FIG. 10. Side elevation and end elevation drawing of testing structure



ment of the truss at panel points was obtained. Lateral securing of the upper chords was provided by the 8 in., 11 1/2 lb. channel purlins which were bolted to the purlin clips, mounted on the platform saddles at each panel point. The gantry ends of the purlins were bolted to clip angles, welded to an 8 in. by 1/2 in. by 24 in. slip plate, in turn bolted to the inside face of the gantry posts. The lower purlin flange bolt, only, was drawn tight, the web bolt nuts being made finger tight only, to permit the purlins to rotate freely about the lower flange connection. The slip plate bolt holes were slotted to permit of vertical adjustments as truss deflection developed. The edges of the purlins at the panel points were beveled to prevent binding, as the truss deflected vertically.

The platform saddles from which the platform suspension rods were hung were mounted with the purlins. The lower chord of the truss was sway braced by two 3/4 in. round rods passing through the gusset plates of joints 5 and 5'. The outboard ends of these rods were bolted to planking, nailed across the gantry posts. Either side of the gusset plates, 1 in. pipe sleeves placed over and tacked to these rods prevented movement; at the gentries the rod-hole was slotted to permit of vertical adjustment as the truss chord deflected. Wherever any lateral attachments were provided, means for vertical adjustment was also provided, which allowed for leveling with the truss when necessary.

Details of Testing Structure

FIG. 11 (*right*). Method of securing the truss at the rigid anchorage

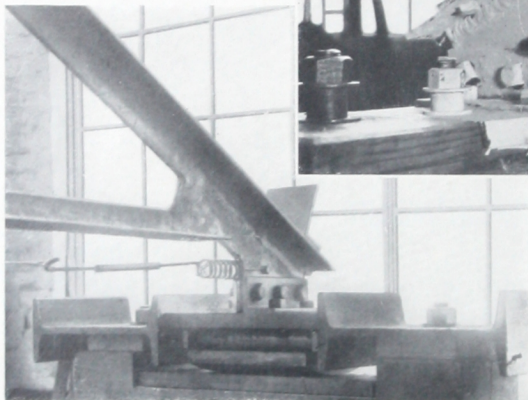
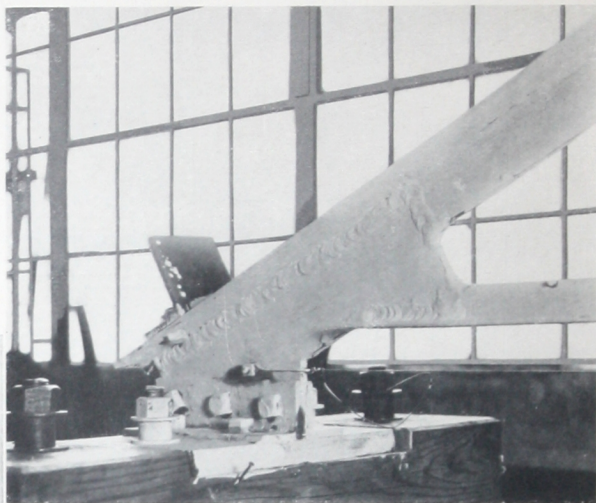


FIG. 12 (*left*). The free end employed a set of rollers mounted on a rocker casting between a lubricated I-beam race

FIG. 13 (*right*). Condition of the upper chord in the third panel (Case 6) after collapse of this member had occurred



The free end of the truss was restrained laterally by two sections of 18 in. I-beam (Fig. 7) bolted to the concrete support, either side of the bearing plate, the inside flange of which provided a horizontal race for the truss bearing plate. The points of contact were lubricated with grease to insure free movement.

TESTING METHODS

A typical testing procedure, after erection of truss and mounting of testing accessories, applicable to all cases in general, follows:*

- (1) The second set of strain gauge readings was taken, representing the true dead load stresses resulting from all weights sustained by a truss with the exception of wooden platforms.
(The first set of strain gauge readings was taken before the trusses were mounted in the test structure.)
- (2) Base vertical and horizontal readings taken.
- (3) Truss painted with white portland cement. After this coating dried, the drilled strain gauge stations were exposed, cleaned and lubricated.
- (4) The first load increment of pig iron aggregating, with the dead load, 3000 lb. per platform, was placed.
- (5) Purlins leveled.
- (6) Vertical and horizontal deflection readings taken.
- (7) The third set of strain gauge readings was taken, representing the true stresses at normal or work load of truss.
- (8) The second load increment of 3000 lb. of pig iron was placed.
- (9) Purlins leveled.
- (10) Vertical and horizontal deflection readings taken.
- (11) The fourth set of strain gauge readings taken.
- (12) The third load increment of 3000 lb. of pig iron was placed.
(In some cases this third load increment was divided into 1000 lb. and 2000 lb. sub-increments for the purpose of procuring deflections and for strain development observations.)
- (13) Purlins leveled. (The riveted truss and the tubular truss failed at or before this point.)
- (14) Deflections read.
- (15) The fifth set of strain gauge readings was taken.
- (16) The fourth load increment of pig iron was placed. This increment approximated 1000 lb. by sub-increments of 500 lb. with the exception of Case 6, which failed before the 1000 lb. fourth increment was applied.

The pig iron which was used to weigh the trusses was laid consecutively on each platform, starting with No. 1, and proceeding to No. 7, thence back to No. 1, platform, etc., a pig at a time. All recorded test data was witnessed by, and reconciled with, representatives of the Pittsburgh Testing Laboratory.

*NOTE: The dead comparable weight of each truss was determined by weighing a completed welded truss on a certified 2,000 lb. portable platform scale. Certification for both this scale and the 2,000 lb. counter scale by the Bureau of Weights and Measures at Buffalo was made. The weight of purlins, platforms and accessories, and all attachments to the truss, causing increments of load on an erected truss, were accurately weighed before mounting.

Condition of Joint No.

All trusses were painted with one coat of white portland cement for the purpose of increasing visibility of the Lüders lines. These so-called Lüders lines appear when the stresses approach the yield

FIG. 14 (*right*). Joint I of Case 1 at free end showing slight rupture in insert plate which developed at a load of approximately 63,000 lb., which point was reinforced as shown in Fig. 16

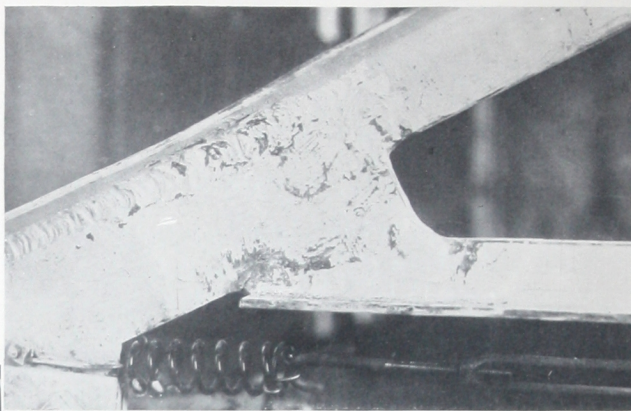


FIG. 15 (*left*). Joint I of Case 2, showing close-up of the upper chord of the riveted truss adjacent to the free end showing development of high stresses in the vertical leg of upper chord as evidenced by the Lüders lines

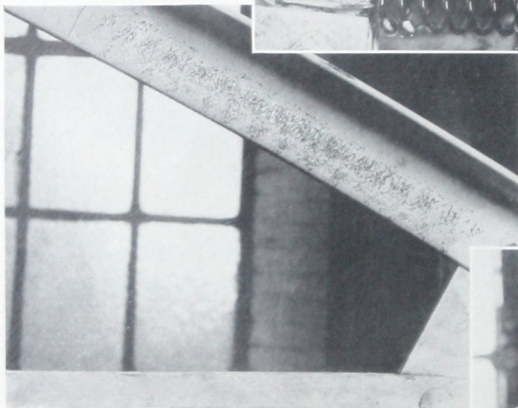


FIG. 16 (*right*). Joint I at the rigid end of Case 3, showing effect of hammer test applied at each joint of each truss when twice the design load, or 6,000 lb. per panel, or 42,000 lb. per truss, was sustained. After test was applied, during which no joint failures developed, the cement coating was re-applied for stress observation purposes at the higher load



int No. 1 After Test

point of the material, and are formed by the flaking of the surface oxide, or mill scale. In this case they are accentuated by the contrast with the undisturbed white cement coating.

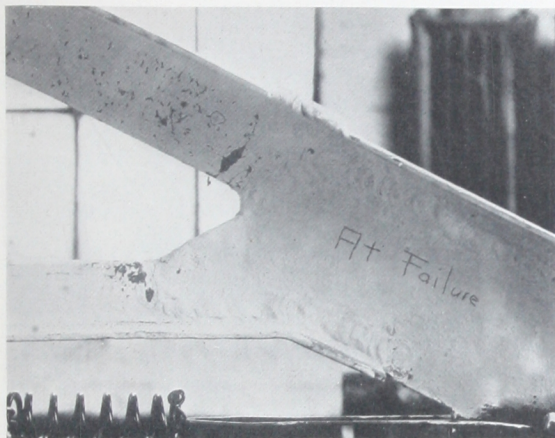


FIG. 17 (left). Joint I at the free end of Case 4 after the upper chord had collapsed. The disturbance in the cement coating indicates the stress intensity in the vicinity of this joint, which, it is noted, is beyond the insert plate and the welded joint. Bending stresses have developed through the section as a result of the rotation of the structure about the free end support.

FIG. 18 (right). Joint I at the rigid end of Case V (tubular truss) showing the splendid condition of this joint after the upper chord had failed. The disturbance in the cement coating in the upper chord adjacent evidences that the maximum strength of this member was rapidly being approached at the time failure occurred.

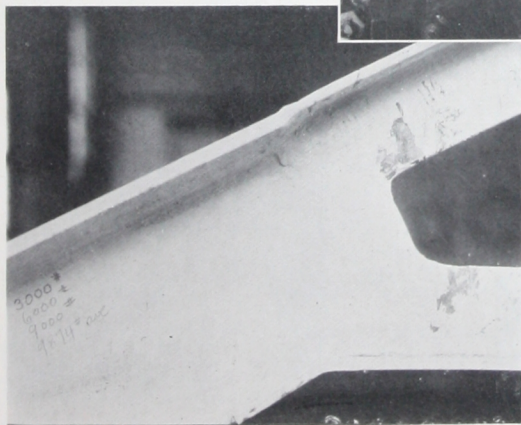
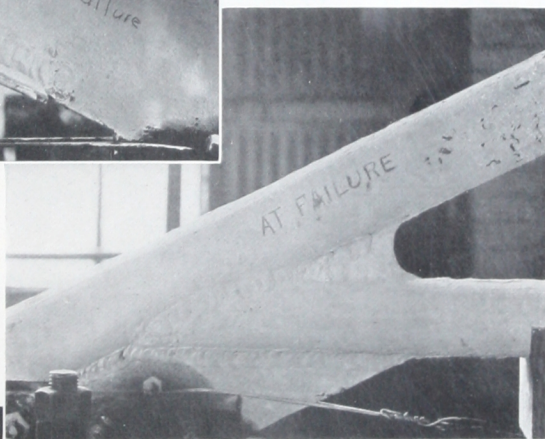


FIG. 19 (left). Joint I at the free end of Case 6, showing typical condition of these joints where failure of the upper chord occurred. Attention is again called to the undisturbed condition of the weld joining the members to the plate, in this case being an integral part of a rolled section made up from a 15-inch I-beam (see drawing Plate VI, Joint I)

Column Failures

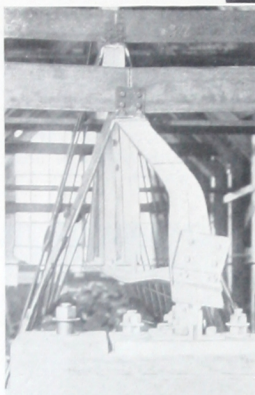


FIG. 20. CASE 1

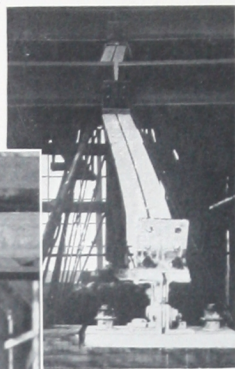


FIG. 21. CASE 2

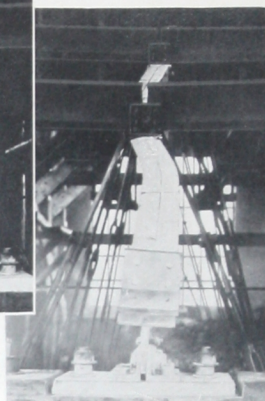


FIG. 22. CASE 3

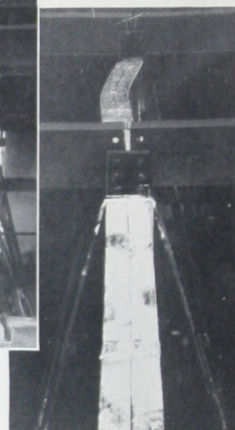


FIG. 23. CASE 4



FIG. 24 (left).
CASE 5

FIG. 25 (right).
CASE 6



General Summary of Results

WHILE there were no joint failures with either type of construction, a study of the results of these full scale tests credit the welded trusses with a uniformly greater stability than a truss of equal weight constructed with riveted joints.

Of particular interest are the results of the tests of Cases 3 and 6. The Case 3 having identical member design and weighing slightly less than the riveted truss, developed a factor of safety approximately 15 per cent greater. The Case 6, weighing nearly 22 per cent less than the riveted truss, developed a factor of safety of 10 per cent greater.

This work then indicates that a true economic design, or one which uses material in a manner that will provide for uniform strength of structure, will effect a saving of approximately 30 per cent in weight of material.

There are several factors in welded construction that make for economy in weight, the two principal ones being greater rigidity of built-up compression members and the use of net metal for the tension members.

Results of the tests consistently showed that the fixity of the upper chords, both at the panel points and at the stitch straps mid-span, operated to improve the compressive resistance of these members.

The use of net metal is made possible by the high efficiency of stress transfer in the insert plate, which can be held to a minimum of 100 per cent by the proper calculation of the shear welds.

Welding also affords a means of fabricating the small sub-struts into the structure and in their most effective position. A slight saving in weight is also gained by the use of smaller gusset or insert plates.*

That welding does not inject negative factors in a steel structure designed along the lines shown herein, has been consistently proven by the results of the tests and the strain gauge work. The measured stresses across the joints and the stress distribution in the members as determined by the Berry Strain Gauge, conformed very closely to the theoretical joint stresses and the indicated member stress distribution for pin-connected structures.

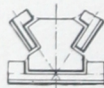
From a welding standpoint the tests brought out that the method employed conformed satisfactorily to all the requirements for stable construction and sound welding practices. Welding was accomplished without difficulty and alignment of the finished structures was attained by the observance of a few simple procedures during the preliminary assembly of each case.

*It was found that for best shop economy, tension members should no doubt be made up of structural tees. These shapes require only a single operation in regard to layout and cutting as compared to twice the amount required for two angles otherwise generally used. Further economy can no doubt be gained where Procedure Control welding conditions prevail, by substituting single vee welding in the place of double vee welding used in these tests.

For a resume of the fabrication costs of the welded trusses, the reader is referred to Appendix E.

CONCLUSION

Summing up, then, it would appear that this method of fabricating roof trusses will effect a saving in all members and parts. Dependable joint strength has been proven by the fact that not one failure occurred in six hundred or so welds involved. While credit for this is due in part to the design for welding, in the most part it is due to observance of the proper welding Procedure Control.



Appendix

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Appendix A

INVESTIGATION OF STRESS DISTRIBUTION

THE stress distribution throughout each truss was determined with a 2-in. and an 8-in. Berry strain gauge. The former records the strain in the metal to within accuracies of plus or minus .00005 in., the latter to within .000025 in., corresponding to a stress of 1500 lb. per sq. in. and 750 lb. per sq. in. respectively. The stresses were measured in the members midway between panel points. A few readings were also taken across the welds and over rivets. For accurately marking the strain gauge measurements, holes were made in the members with a No. 56 drill, established 2 in. and 8 in. apart, carefully reamed and lubricated to keep out particles of the cement coating. Gauge stations were placed close to the center of gravity of each member, and on the extreme fibre on the critical members. Three readings were taken at each station, which insured accurate averages.

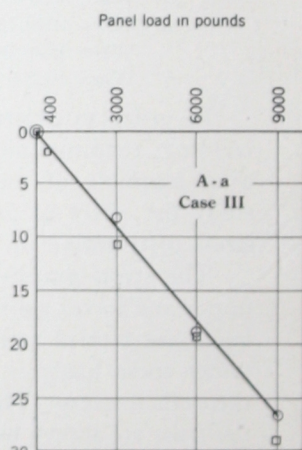
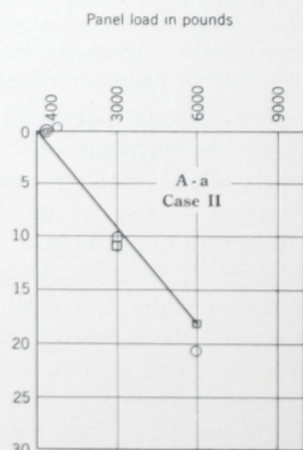
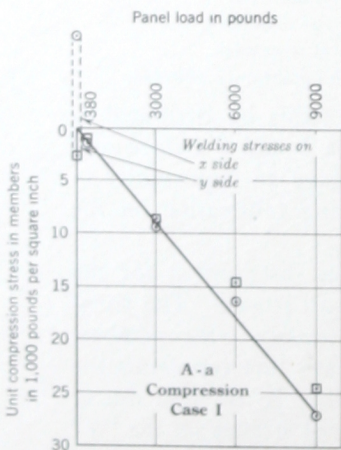
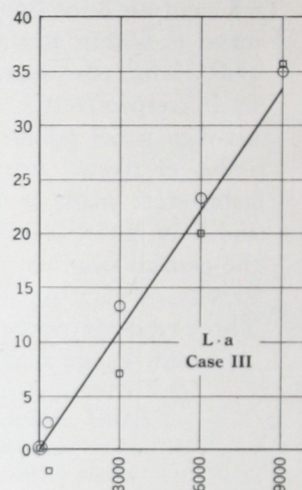
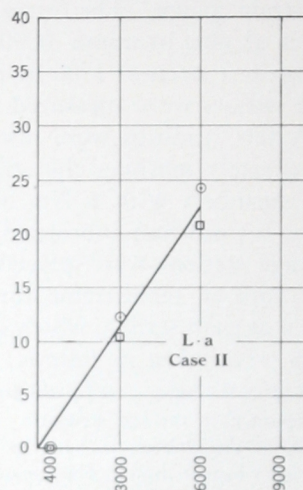
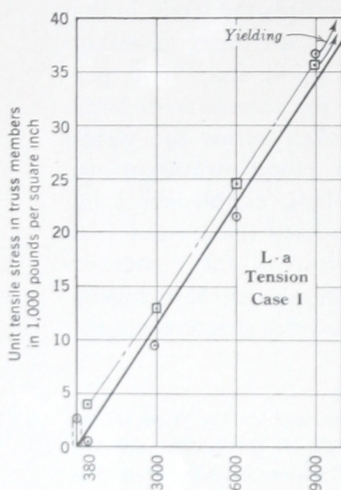
Strain gauge readings were taken as follows:

1. With truss supported at the lower chord panel points.
2. After truss was mounted in the test structure. The load on the truss at this point consisted only of the truss itself, half the weight of the purlins, and the weight of the platform hanger irons. The measured stresses include any stresses set up by erection and aligning.
3. When 3000 lb. per platform, or the normal load, was suspended.
4. When 6000 lb. per platform, or twice normal load, was suspended.
5. When 9000 lb. per platform, or thrice normal load, was suspended.
6. When 10,000 lb. per platform was suspended, where opportunity afforded.

In order to determine the stresses that could possibly be attributed to welding, readings were taken on members of Cases 1 and 3 before and after assembly of the truss. These readings also gave an opportunity to study the effect of any internal stresses set up during fabrication on the stress distribution in the structure under load.

The stress graphs on pages 52-53 show the stress distribution for the upper and lower chord members of the first panel of all trusses. The panel loads are plotted as abscissa and the measured unit stresses in the members under these loads as ordinates. The continuous heavy line represents the theoretical stress. The average measured stresses on the "X" or front side of the truss are shown by the circles, the average measured stresses on the "Y" or opposite side by the squares. The stresses due to welding, determined by measurements taken before and after welding, are plotted on the zero abscissa. The graphs show that the actual stresses agree very closely with the theoretical. In the sub-members the agreement was not as close as in the main chords of the truss, being consistently lower than the theoretical. These sub-members, however, were not subjected to very high stresses under load.

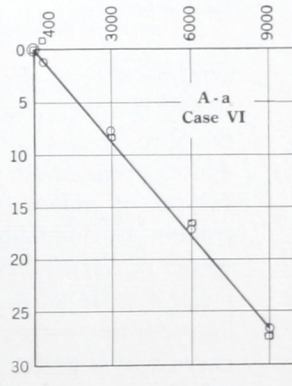
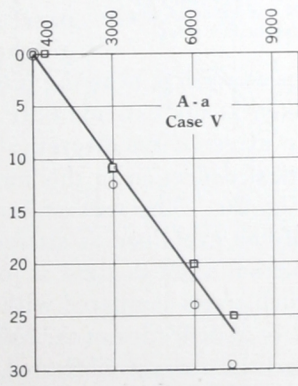
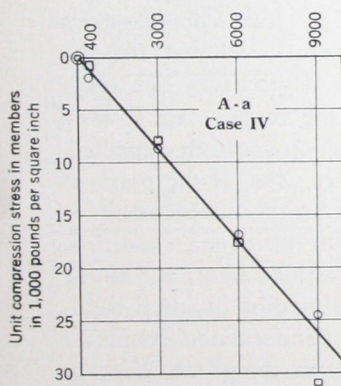
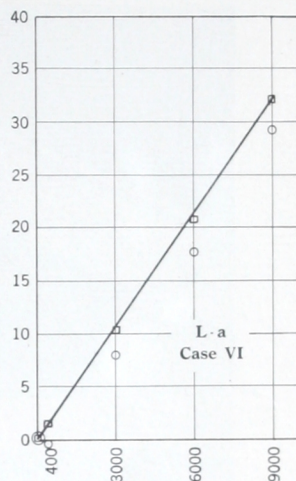
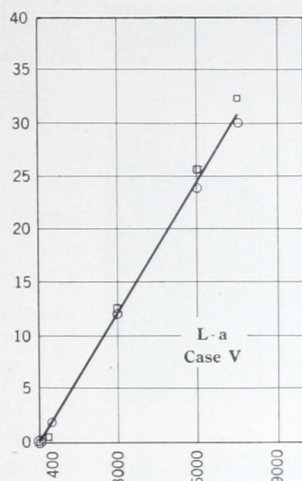
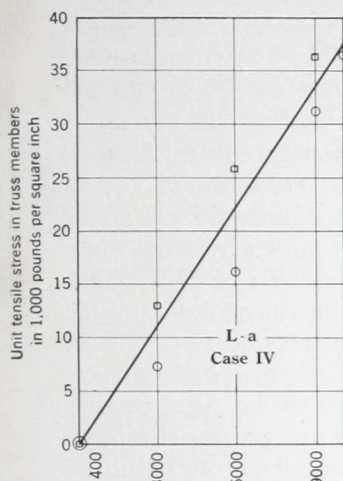
Stresses in Bottom and Top Chord of First



Stresses due to truss loads

- Theoretical unit stress in members
- ○ Average measured unit stresses in members of one (x) side
- □ Average measured unit stresses in members on other (y) side

of First Panel at Fixed End of Truss



Stresses due to truss loads

- Theoretical unit stress in members
 ○ Average measured unit stresses in members of one (x) side
 □ Average measured unit stresses in members on other (y) side

The stresses due to welding are variable in amount and appear to be due to slight bending, as the measurements record tension on one side and compression on the corresponding opposite side. In most cases, these secondary stresses do not exceed 3000 or 4000 lb. per sq. in. They do not influence the stresses due to loading as far as can be determined by the gauge readings.

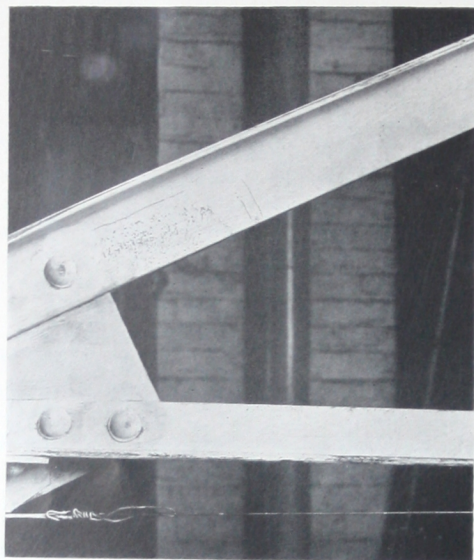


FIG. 26. Joint I at rigid end of Case 2, showing Lüders lines developed adjacent to the plate, indicating presence of bending stresses which are expected due to rotation of the structure about this point

extreme fibre of the legs as well as near the center of gravity. The reading of member *A-a*, taken at the extreme fibre of the vertical leg, showed the fibre stresses to be in excess of the theoretical, which could be expected, due to pronounced vertical deflection in this member, *A-a*, at the point of failure, as shown in Fig. 26. The weaker condition of this member, or column, can undoubtedly be explained as attributable to its end condition, which in riveted construction is not as rigid as in a welded joint, and to the lower stability of the stitch rivets compared with the method of stitch welding, resulting more readily in deflection of the column under load. A special curve which plotted the local stresses measured across the stitch rivet in member *L-a*, showed that the stress in the member at this point is increased in proportion to the decrease in area, due to the rivet hole.

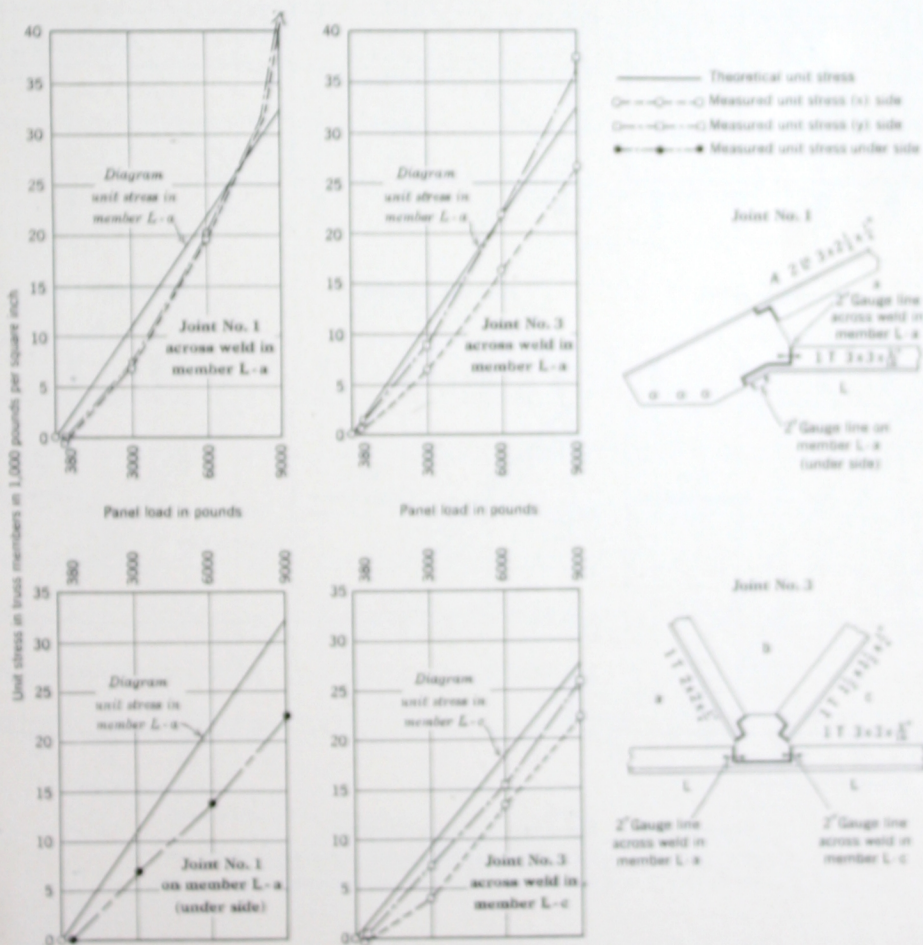
The stresses in the pipe truss, Case 5, were measured at the sides of the pipe and consequently include any bending stresses that may have been set up in the members due to welding or erection in the test structure. Indications of bending stresses were found when measuring the stresses at

The stress graph for Case 2, the riveted truss, shows a total panel load of only 6000 lb., since this truss completely failed while preparations were being made for strain measurements just after the third load increment of 9000 lb. per platform had been placed. The plate shows general conformity of the measured stresses with the theoretical; the measured stresses in the sub-members, however, did not check as closely with the theoretical as is also the case with the welded trusses. For panel *A-a*, top chord, stress graphs were developed for the

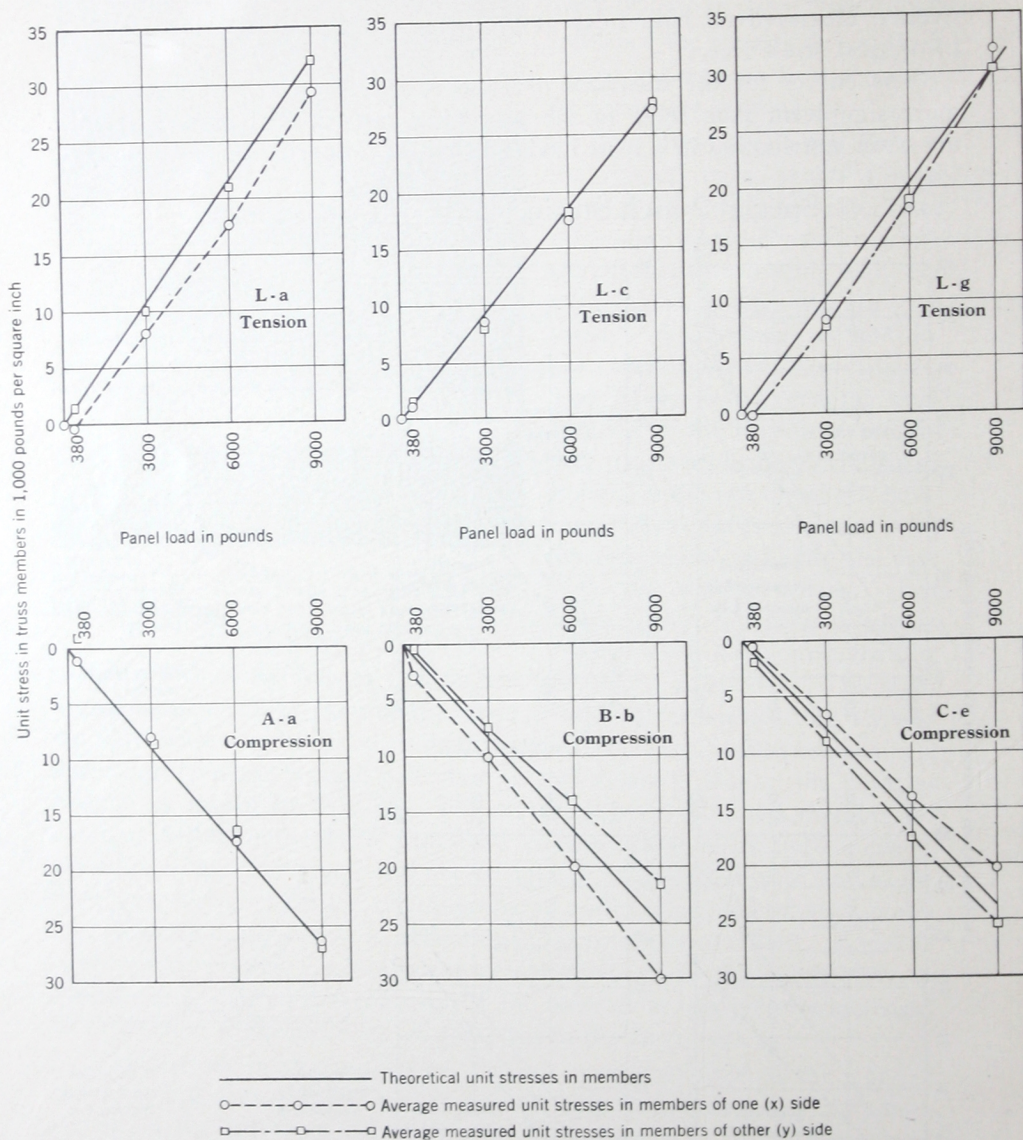
the 350 lb. or dead load increment. On one side of the pipe, these measured stresses were in compression, while on the opposite side they were in tension and of like intensity. Apparently, these bending stresses do not materially influence the strength of the members, since the average of the stresses, measured on both sides of the members, agreed closely with the theoretical stress line.

The curves for the members of Case 6, the tee truss, are somewhat more significant than those in the preceding cases in that they represent the stress distribution in a truss having members proportioned more closely

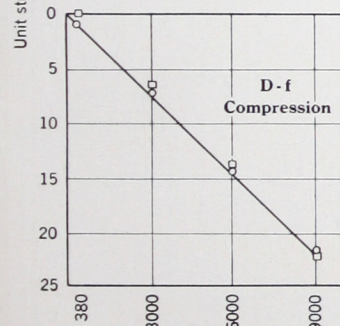
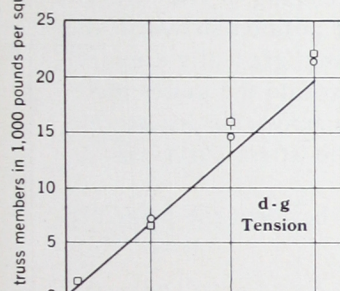
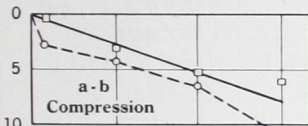
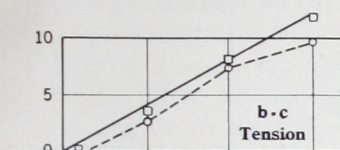
Special 2-inch Strain Gauge Measurements



Stress in Members of So-Called Welded



Welded Tee Trusses (Case 6)

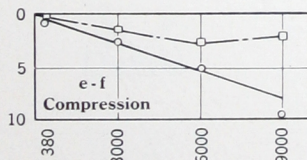
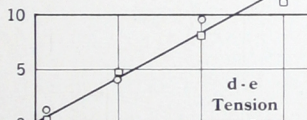
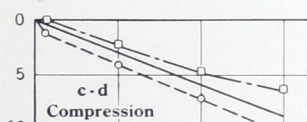


Panel load in pounds

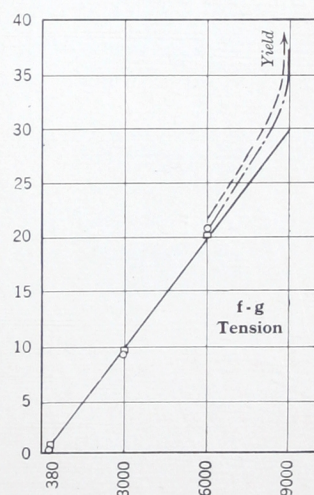
Member		Area sq. in.	Design I. D. S. lb.	Load unit stress lb./sq. in.
A-a	2 L 3 x 2 1/2 x 1/4	2.64	+	23500
B-b	"	"	+	22140
C-e	"	"	+	20790
D-f	"	"	+	19440
L-a	1 T 3 x 3 x 5/16	1.95	-	21000
L-c	"	"	-	18000
L-g	1 T 2 1/4 x 2 1/4 x 1/4	1.19	-	12000
b-c, d-e	1 T 1 1/2 x 1 1/2 x 1/4	0.73	-	3000
a-b, e-f	1 T 2 x 2 x 1/4	1.05	+	2700
c-d	2 L 2 x 2 x 1/4	1.88	+	5370
d-g	1 T 1 3/4 x 1 3/4 x 1/4	0.91	-	6000
f-g	"	"	-	9000

(Note: For truss diagram see other cases)

Design load W=3000 lb.



Panel load in pounds



Panel load in pounds

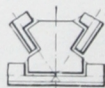
— Theoretical unit stresses in members
 ○ — ○ Average measured unit stresses in members of one (x) side
 ○ — ○ Average measured unit stresses in members of other (y) side

to the theoretical requirements. The lower chord was reduced in size between joints 5 and 5', the main strut *c-d* is smaller, as are the sub-strut and web tension members.

To determine the stresses across the welds at critical points, special readings were taken on Truss 6, at joints 1 and 3. The location of these measurements and the result are shown on pages 56-57. At joint 1, it will be observed that the stresses across the vertical leg tension weld agree closely with theory up to the time where relatively high loads were applied on the truss, at which time bending stresses are imposed by the deflections of member *L-a*. These stresses were clearly manifested in the base metal of the member adjacent to the weld, by general disturbance in the cement coating, since the yield point of the metal had been reached and passed. The strain gauge recorded these stresses up to the yield point, permitting indication on the stress graphs as shown. The weld remained intact, judging from the condition of the cement coating. The stresses, measured on the under side of member *L-a* at the position shown in the sketch, which are made up of direct and bending stresses, gradually diminish in intensity, indicating transfer of the member stress into the gusset plate.

At joint 3, the average of the measured stresses across the leg tension welds agrees fairly closely with the theoretical unit stress curves, being somewhat less at both gauge stations.

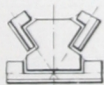
The result of these joint readings would seem to bear out the principle of stress transfer as explained previously.



Appendix B

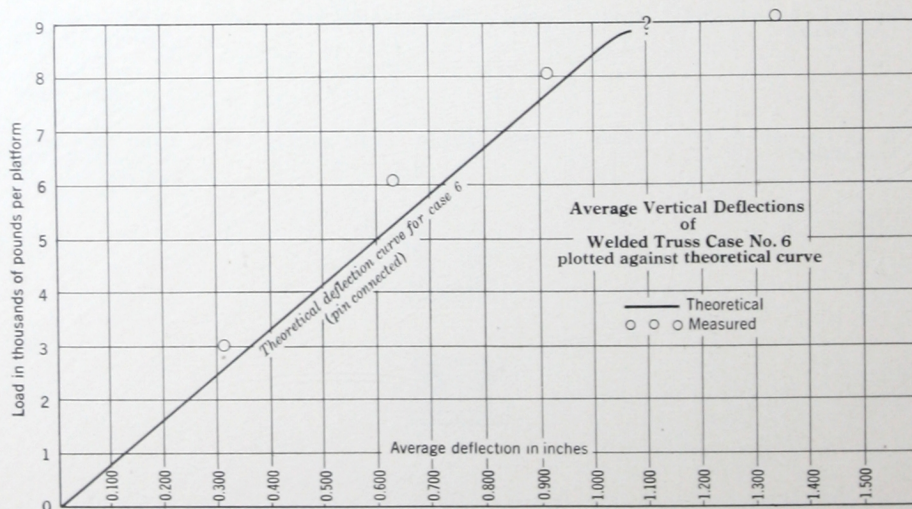
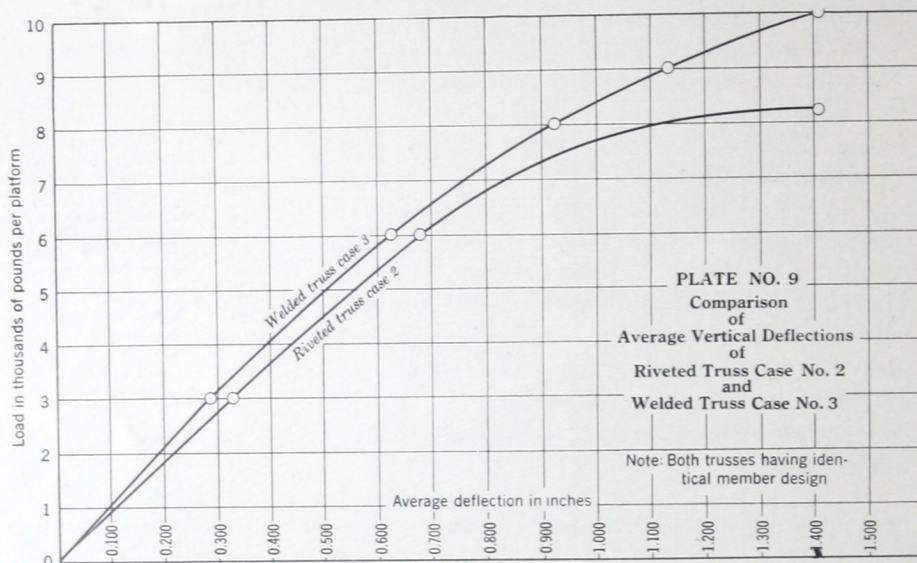
COMPARISON OF FACTORS OF SAFETY AND DEFLECTIONS

CASE	Weight of Truss	Total Suspended Load	Factor of Safety	Total Dead Load at Which Zero Deflections Were Made	MAXIMUM DEFLECTIONS			REMARKS
					Vertical		Horizontal	
					Rigid End	Free End		
No. 1—Preliminary Truss Welded	1199	71,988	3.428	2646.25	1.5	1.531	Above .750	Last horizontal reading at 63,000 lb. load. Vertical deflection read just before failure. Upper chord failed in first panel from rigid end.
No. 2—Riveted Truss	1161	63,037	3.00	2611.00	1.42	1.33	.297	Deflections read just before failure at approximately 8200 lb. per panel. Lower chord touching gauge wire, rigid end.
No. 3—Duplicate of Riveted Truss but with Weld Joints	1079	72,214	3.439	2541.00	1.37	1.37	.593	Upper chord failed in first and second panels from rigid end.
No. 4—Symmetrical Welded Truss with Double Angle Members	1079	70,291	3.347	2541.00	1.45	1.52	.562	Deflections read just before upper chord failed.
No. 5—Tubular Truss Welded	916	62,125	2.958	2397.50	1.04	1.17	.664	Deflections taken at approximately 8670 lb. per panel. Upper chord failed in first panel from free end.
No. 6—T-member Truss Welded	912	69,205	3.295	2393.00	1.283	1.313	.656	Upper chord failed in third panel from free end.



Appendix C

DEFLECTION GRAPHS



Appendix D

TIME DATA FOR FABRICATION OF TRUSSES

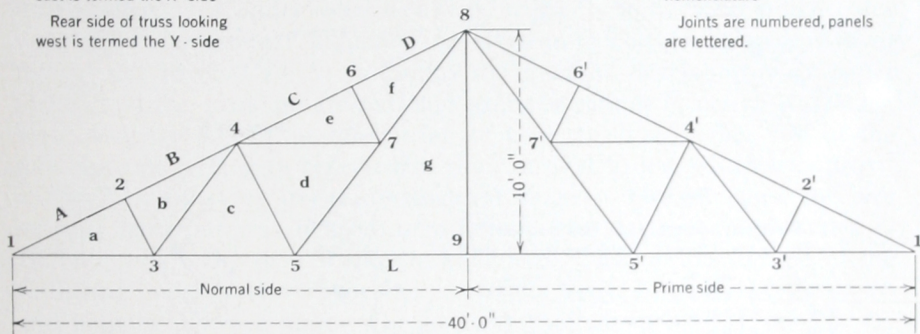
THE accompanying drawings and table show the welding time required for each joint of each truss. The drawings show the welding applications at each joint of Case 3. The first sketch is a key drawing for locating the numbered joints as used in the table.

Front side of truss looking east is termed the X-side

Rear side of truss looking west is termed the Y-side

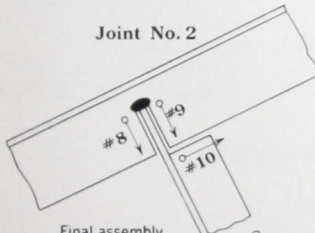
Outline of truss showing general nomenclature

Joints are numbered, panels are lettered.



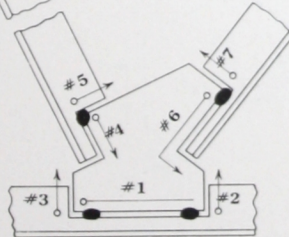
Time Data		Joint No. 1	
Preliminary assembly		Final assembly	
Welds Nos. 1, and 2		Welds Nos. 3, and 4	
X-N	19 min. □	11 min. *	
Y-N	17 min. □	6½ min. *	
X-P	24 min. □	10 min. *	
Y-P	14 min. □	9½ min. *	

Joint No. 2



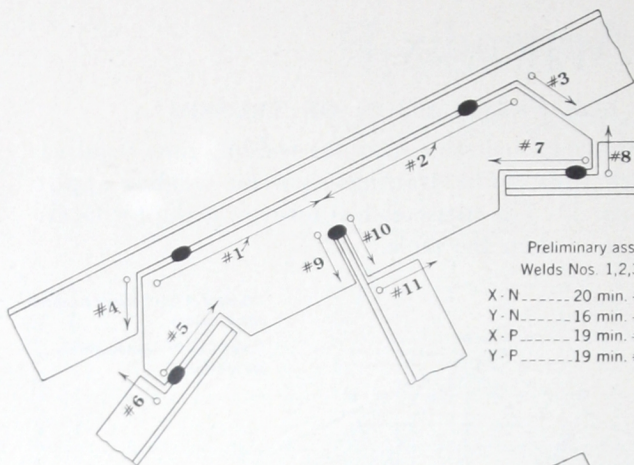
Final assembly	
Welds Nos. 8, 9, and 10	
X-N	5 min. *
Y-N	4 min. *
X-P	5 min. *
Y-P	7½ min. *

Final assembly	
Welds Nos. 8, 9, and 10	
Joint No. 6	
X-N	7 min. □
Y-N	3½ min. □
X-P	6 min. □
Y-P	8 min. □



Joint No. 3

Time Data	
Preliminary assembly	
Welds Nos. 1, 2, and 3	
X-N	5 min. *
Y-N	4 min. *
X-P	9 min. □
Y-P	7½ min. □
Final assembly	
Welds Nos. 4, 5, 6, and 7	
X-N	9½ min. *
Y-N	11 min. *
X-P	10 min. *
Y-P	12 min. *



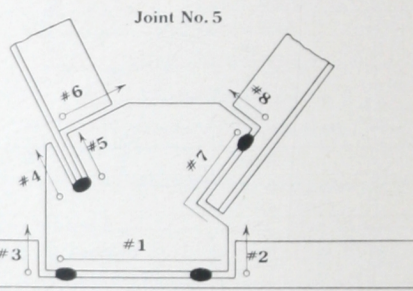
Joint No. 4

Time Data

Preliminary assembly		Final assembly	
Welds Nos. 1, 2, 3, and 4		Welds Nos. 5, 6, 7, 8, 9, 10, and 11	
X · N	20 min. *	20	min. □
Y · N	16 min. *	24	min. *
X · P	19 min. *	14½	min. □
Y · P	19 min. *	21	min. *

Time Data

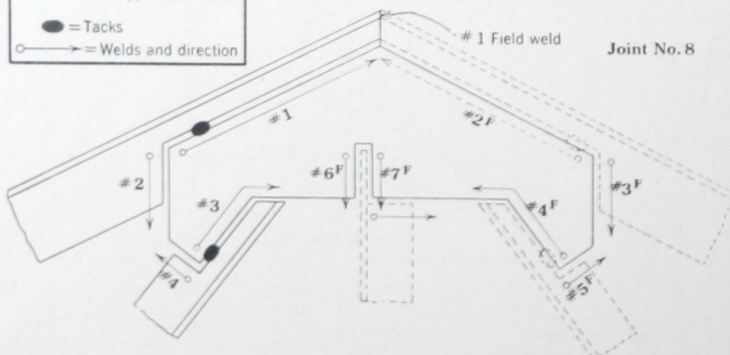
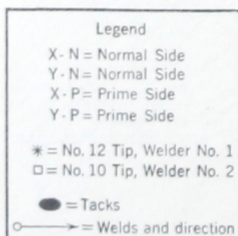
Preliminary assembly		Final assembly	
Welds Nos. 1, 2, and 3		Welds Nos. 4, 5, 6, 7, and 8	
X · N	8 min. *	11½	min. *
Y · N	6 min. *	19	min. □
X · P	6½ min. *	14	min. *
Y · P	8½ min. *	13	min. □



Joint No. 5

Time Data

Preliminary assembly		Final assembly	
Welds Nos. 1, and 2		Welds Nos. 3, and 4	
X · N	10½ min. *	8	min. □
Y · N	12 min. *	8	min. □



Joint No. 8

Appendix E

COST STUDIES

THROUGHOUT the fabrication operations, careful cost accounting was made. The items of acetylene, oxygen, rod and labor consumed were recorded for each detail operation for each welded truss, from which data, the cost summary given in Table I was obtained.

These cost figures, of course, represent that of single unit construction conducted under rather inefficient shop methods, unassisted by the devices usually developed under what would be termed regular shop practice for welding. The gradual reduction of fabricating costs, with exception of the tubular truss (Case 5) is due to the gradual development of better methods as the work progressed, and to modifications in design warranted by experience gained in the testing of Case 1. The higher cost of the tubular truss is largely reflected in the cost of the pipe, which is approximately twice that of standard structural shapes. For all cases, the cost of truss materials was figured at mill base plus carload freight rates to Buffalo, where the shop work was conducted. Labor was figured in the following hourly rates: layout men, \$1.00 per hour; welders, 80c per hour, and helpers, 50c per hour. Oxygen was entered at \$0.0125 per cu. ft., acetylene at \$0.013 per cu. ft., and rod at 20c per lb.*

To those familiar with the shop costs for this type of truss, the figures will provide some basis for comparison with more conventional methods. We must wait for an opportunity to fabricate a group of welded trusses, where the economies of multiple operation will be felt, to obtain good average costs.

*The above unit costs, of course, may or may not conform to prevailing costs in a fabricating shop at the present time. For this work the figures were arbitrarily determined in order to consummate the cost analysis.

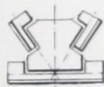


TABLE I

ITEM	CASE 1	CASE 3	CASE 4	CASE 6	TUBULAR TRUSS CASE 5
1. Shop Layout	2.85	2.00	2.50	1.50	5.38
2. Shop Cutting	5.11	4.54	6.28	2.86	4.73
3. Preliminary Assembly	48.76	15.00	14.71	9.87	12.99
4. Final Assembly		15.68	16.14	16.64	21.33
5. Shop Handling	.90	.81	.81	.69	.69
6. Painting, \$1.40 per ton.....	.84	.75	.75	.64	.63
7. Steel (delivered to Welding Dept.)	30.44	25.97	26.47	23.04	39.20
TOTAL DIRECT	88.90	64.75	67.66	55.24	84.95

EXPLANATION OF COST SUMMARY ITEMS

Item 1—Shop Layout:

Layout of gusset plates and members preparatory to shop cutting is further explained under "Fabrication of Welded Trusses."

Item 2—Shop Cutting:

Shop oxy-acetylene cutting of gusset plates and members to conform to the template profiles.

Item 3—Preliminary Assembly:

This item exists for Cases 3, 4, 5 and 6, and is explained in detail under the heading of "Fabrication of Welded Trusses" for these cases. In general it consists of assembling the gusset plates with their respective principal members and making the welds at the contacting joints.

Item 4—Final Assembly:

This consists of the assembly of all truss members into a single truss or half truss section and making the remaining welds. Involved in this operation is the clamping or tacking of the members at the joints prior to making the final welds and correcting any discrepancies in alignment as a result of the final welding.

Item 5—Shop Handling:

Represents the cost of handling the truss materials during the process of fabrication in the shop.

Item 6—Painting:

This item is included for the purpose of completing the cost analysis and is based on an arbitrary value of \$1.40 per ton, which may or may not be correct for lightweight trusses. The results show comparative costs on a weight basis. The proportional values should not be much in error. The trusses involved in this test were not painted.

TABLE II. SHOP LAYOUT

DETAILS OF SHOP LAYOUT	CASE No. 1		CASE No. 2		CASE No. 3		CASE No. 4		CASE No. 5		CASE No. 6		REMARKS
	Time	Cost	Time	Cost	Time	Cost	Time	Cost	Time	Cost	Time	Cost	
GUSSET PLATES													
Layout man @ \$1.00 per hour..	20 min.	\$.34			25 min.	\$.4167	20 min.	\$.333	5 min.	\$.0833	5 min.	\$.0833	
Helper @ .50 per hour.....					do	.2085	do	.166	do	.0417	do	.0417	
Total layout cost of Gusset Plates		\$.34				\$.6252		\$.500		\$.1250		\$.1250	
MEMBERS													
Layout man @ \$1.00 per hour..	100 1/4 min.	\$1.675			55 min.	\$.9168	80 min.	\$1.333	3 h. 30 m.	\$3.5000	55 min.	\$.9168	
Helper @ .50 per hour.....	do	.838			do	.4587	do	.666	do	1.7500	do	.4587	
Total layout cost of Members..		\$2.513				\$1.3755		\$2.000		\$5.2500		\$1.3755	
Grand Total		\$2.85				\$2.00		\$2.50		\$5.375		\$1.50	

TABLE III. SHOP CUTTING

DETAILS OF SHOP CUTTING AND CLEARING	CASE No. 1		CASE No. 2		CASE No. 3		CASE No. 4		CASE No. 5		CASE No. 6		REMARKS
	Time or Cu. Ft.	Cost	Time or Cu. Ft.	Cost	Time or Cu. Ft.	Cost	Time or Cu. Ft.	Cost	Time or Cu. Ft.	Cost	Time or Cu. Ft.	Cost	
GUSSET PLATES													
Cutter @ 80c per hr.....	99 min.	\$1.200			56 min.	\$.7470	55 min.	\$.7337	30 min.	\$.400	24 min.	\$.3202	*One helper serving two operators. Helper's rate 50c per hour.
*Cutter Helper @ 25c per hr....	do	.4128			do	.2335	do	.2293	do	.125	do	.1001	
Oxygen @ \$.0125 per cu. ft....	55 cu. ft.	.6875			72 cu. ft.	.9000	97 cu. ft.	1.2125	31.00	.3875	29.75	.3712	
Acetylene @ \$.0130 per cu. ft....	21.35 cu. ft.	.2775			211 cu. ft.	.2795	29 cu. ft.	.3770	9.20	.1196	7.70	.1001	
Total Cutting Cost of Gusset Plates		\$2.6978				\$2.1600		\$2.5525		\$1.0321		\$.8916	
MEMBERS													
Cutter @ 80c per hr.....	71 1/4 min.	\$.9538			49 1/2 min.	\$.6603	77 min.	\$1.0272	10 1/2 min.	\$1.400	4 1/2 min.	\$.5803	** Done by helper during cutting period incl. above.
*Cutter Helper @ 25c per hr....	do	.2594			do	.2438	do	.4111	do	.3178	1 1/2 min.	.1904	
Oxygen @ \$.0125 per cu. ft....	42.8 cu. ft.	.5350			73 cu. ft.	.9125	112 cu. ft.	1.4000	10.00	1.250	55.25	.6904	
Acetylene @ \$.0130 per cu. ft....	18.2 cu. ft.	.2366			22 cu. ft.	.2860	34 cu. ft.	.4420	30.60	.3978	14.30	.1859	
Total Cutting Cost of Members..		\$2.0235				\$2.0652		\$3.1903		\$3.4856		\$1.6382	
CLEARING													
Gusset Plates													
Helper @ 50c per hr.....	19 min.	\$.158			**		37 min.***	\$.3086	5 min.	\$.0417	13 min.	\$.1084	*** Laminated sheets prevent chipping. Sheets could be approx. 15 minutes.
Members													
Helper @ 50c per hr.....	27 1/2 min.	.230			38 min.	\$.3169	28 min.	.2335	20 min.	.1668	26 min.	.2168	
Total Clearing Cost, Plates and Members		\$.388				\$.3169		\$.5421		\$.2085		\$.3252	
Grand Total		\$5.11				\$4.54		\$6.28		\$4.73		\$2.855	

TABLE IV. PRELIMINARY ASSEMBLY

	CASE NO. 1		CASE NO. 2		CASE NO. 3		CASE NO. 4		CASE NO. 5		CASE NO. 6		REMARKS
	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	
SETTING-UP, CLAMPING AND TACK ING AND STITCH WELDING													
Lay-out man @ \$1.00 per hr.....					40 min. do	\$.6668 .3336	50 min. do	\$.8335 .4170	60 min. do	\$1.000 .500	43 min. do	\$.6168 .3586	
Helper @ .50 per hr.....					22 min. 51 cu. ft.	.2935 .6375	24 min. 48½ cu. ft.	.3202 .6063	20 min. 22 cu. ft.	.2668 .2750	72 min. 44 cu. ft.	.9604 .5500	
Oxygen @ .0125 per cu. ft.....					48.3 cu. ft. 1.43 lb.	.6305 .2860	46½ cu. ft. .89 lb.	.6013 .1780	24 cu. ft. .834 lb.	.2730 .1668	41.8 cu. ft. .94 lb.	.5434 .1880	
Acetylene @ .0130 per cu. ft.....													
Rods @ .20 per lb.....													
Total						\$2.8479		\$2.9563		\$2.4816		\$3.2172	
WELDING IN GUSSET PLATES													
Welder @ \$.80 per hr.....					254 min. do	\$3.3884 1.0592	23½ min. do	\$3.1149 .9737	247 min. do	3.2949 1.0300	153½ min. do	\$2.0477 .6401	*One helper serving two operators.
*Helper @ .25 per hr.....					183 cu. ft.	2.2875	183 cu. ft.	2.2875	164 cu. ft.	2.0500	113.2 cu. ft.	1.4150	Helper's rate 50c per hour.
Oxygen @ .0125 per cu. ft.....					174 cu. ft.	2.2620	174½ cu. ft.	2.2685	156 cu. ft.	2.0380	107.5 cu. ft.	1.3975	
Acetylene @ .0130 per cu. ft.....					9.8 lb.	1.9600	9 lb.	1.8000	10.55 lb.	2.1100	3.58 lb.	.7160	
Rods @ .20 per lb.....													
Total						\$10.9571		\$10.4446		\$10.5129		\$6.2163	
STRAIGHTENING													
Welder @ \$.80 per hr.....					26 min. do	\$.3468 .2168	30 min. do	\$.4000 .2500	none		20 min. do	\$.2667 .1667	*Included with welding above. Equal to ap- prox. 10 per cent of oxygen and acetylene used.
Helper @ .50 per hr.....					25.5 cu. ft.	.3187	27 cu. ft.	.3375					
Oxygen @ .0125 per cu. ft.....					24.3 cu. ft.	.3159	24½ cu. ft.	.3185					
Acetylene @ .0130 per cu. ft.....													
Total						\$1.1982		\$1.3060				\$.4334	
Grand Total						\$15.00		\$14.71		\$12.99		\$9.87	

TABLE V. FINAL ASSEMBLY

DETAILS OF FINAL ASSEMBLY	CASE No. 1		CASE No. 2		CASE No. 3		CASE No. 4		CASE No. 5		CASE No. 6		REMARKS
	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	Time Cu. Ft. or Pounds	Cost	
CLAMPING AND TACKING													
Layout man @ \$1.00 per hr....	132 min.	\$2.2000			60 min.	\$1.0000	45 min.	\$.7502	60 min.	\$1.0000	50 min.	\$.8335	
Helper @ .50 per hr.....	do	1.1000			do	.5000	do	.3753	do	.5000	do	.4170	
Welder @ .80 per hr.....	108 min.	1.4400			22 min.	.2935	25 min.	.3335	25 min.	.3335	29 min.	.3868	
Oxygen @ .0125 per cu. ft....	170 cu. ft.	2.1250			13½ cu. ft.	.1687	15 cu. ft.	.1875	15.00 cu. ft.	.1875	17.4 cu. ft.	.2175	
Acetylene @ .0130 per cu. ft....	162 cu. ft.	2.1060			12.8 cu. ft.	.1664	14.3 cu. ft.	.1859	14.3 cu. ft.	.1859	16.58 cu. ft.	.2155	
Rod @ .20 per lb.....	3 lb.	.6000			.56 lb.	.1120	1.1 lb.	.2200	.50 lb.	.1000	.50 lb.	.1000	
Total		\$9.571				\$2.2406		\$2.0524		\$2.3069		\$2.1703	
WELDING (Final Shop Application)													
Welder @ .80 per hr.....	894 min.	\$11.9260			319 min.	\$4.2554	324 min.	\$4.3222	428½ min.	\$5.7162	339 min.	\$4.5223	*One helper
*Helper @ .25 per hr.....	do	3.7230			do	1.3302	do	1.3511	do	1.7868	do	1.4136	serving two
Oxygen @ .1025 per cu. ft....	664 cu. ft.	8.3000			233½ cu. ft.	2.9187	223 cu. ft.	2.7875	324½ cu. ft.	4.0562	254 cu. ft.	3.1750	operators.
Acetylene @ .0130 per cu. ft....	630 cu. ft.	8.1900			222 cu. ft.	2.8860	212 cu. ft.	2.7560	309 cu. ft.	4.0170	242 cu. ft.	3.1460	Helper's rate
Rod @ .20 per lb.....	27 lb.	5.4000			7.09 lb.	1.4180	11.2 lb.	2.2400	17.22 lb.	3.4440	8.92 lb.	1.7840	50c per hour.
Total		\$37.5390				\$12.8083		\$13.4568		\$19.0202		\$14.0409	
STRAIGHTENING													
Welder @ .80 per hr.....	45 min.	.6003			20 min.	\$.2668	20 min.	\$.2668	none		20 min.	\$.2668	
Helper @ .50 per hr.....	do	.3753			do	.1668	do	.1668			do	.1668	
Oxygen @ .1025 per cu. ft....	27 cu. ft.	.3375			8 cu. ft.	.1000	8 cu. ft.	.1000			*	*	*Included with
Acetylene @ .0130 per cu. ft....	25.7 cu. ft.	.3341			7.25 cu. ft.	.0943	7.25 cu. ft.	.0943			*	*	welding above.
Total		\$1.6472				\$.6279		\$.6279				\$.4336	
Grand Total		\$48.76				\$15.68		\$16.14		\$21.33		\$16.64	

TABLE VI. WELDING TIME COMPARISON TABLE

JOINT NUMBER	CASE 3						CASE 4						CASE 6						TUBULAR TRUSS CASE 5					
	Preliminary Assembly			Final Assembly			Preliminary Assembly			Final Assembly			Preliminary Assembly			Final Assembly			Preliminary Assembly			Final Assembly		
	X	Y	Side	X	Y	Side	X	Y	Side	X	Y	Side	X	Y	Side	X	Y	Side	X	Y	Side	X	Y	Side
	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.	min.
1. Normal	19	17	10	11	6½	9½	17	17	12½	12	12	12	12	12	10	24	27½	31	35	35	31	35	31	*
1. Prime	24½	14	5	5	9½	4	15½	12½	12	12	12	12	12	10	8	22	23	31	37	37	31	37	25	*
2. Normal				5	4	7½				7	8	8				5½	6	No Insert	3	3	No Insert	3	3	3
2. Prime				5	7½					7	9	9				5½	7	No Insert	3	3	No Insert	3	3	3
3. Normal	5	4		10	9½	12	7	7½	11	10	13	12				8½	8	No Insert	4½	4	No Insert	4½	4	6
3. Prime	9	7½		10	12		7	7½	11	10	12	12				8½	8	No Insert	4½	4	No Insert	4½	4	6
4. Normal	20	16		20	24		20	18	21	21	20	20				17½	17	24						
																		8						
4. Prime	19	10		14½	21		17	14½	18	18	22	22				18½	17	8	36½	36½	21	36½	21	†
5. Normal	8	6		11½	19		8	6	14	14	13	13				10½	11	26	34½	34½	23	34½	23	
5. Prime	6½	8½		14	13		7	7	11	11	12	12				10½	12	20	17½	17½	12	17½	12	
6. Normal				7	3½				6	6	5	5				6	6	31	18½	18½	14	18½	14	
6. Prime				6	8				6	6	7	7				7	7½							
																		Splice and butt weld in upper chord.						
7. Normal				11	8½		6	4	11	11	12	12				7	8							
7. Prime	7½	7		11	10		5	6½	9	9	10	10				10	8½							
8. Normal				8	8		10	14	4	4	6	6				10	11½	16						
8. Prime				8	8		10	14	4	4	6	6				10	11½	16						
																		Field Weld						
REMARKS	*No. 12 tip, Welder No. 1. †No. 10 tip, Welder No. 2.						*No. 12 tip, Welder No. 1. †No. 10 tip, Welder No. 2.						FIELD WELDS No. 8—4½ min. No. 9—6½ min. Splice at Joint No. 5N—22 min. Splice at Joint No. 5P—16½ min. Note normal side welded with No. 12 tip. Prime side welded with Nos. 12 and 10 tips.						Note.—Tension chord insert adjacent to joint 5, which splices with member g-L in field. Normal side 7 min. Prime side 8 min. Done by Welder No. 2 with No. 10 tip. *Normal side 7 min. Prime side 8 min. †No. 10 tip, welder No. 2.					

Appendix F

Physical Properties of Principal Truss Members as Determined by Testing the Coupons
Prepared from the Scrap Material Resulting from the Insert Cuts

MARKING OF COUPON TEST SPECIMENS

No. of Specimens	Marking	Meaning of Marking	No. of Specimens	Marking	Meaning of Marking
4	1-TC 1-BC	Case 1—Top Chord Case 1—Bottom Chord	2	WAMB-BC	Case 3—Bottom Chord
4	1-D-TC 1-D-BC	Case 4—Top Chord Case 4—Bottom Chord	1	T-TC	Case 5—Top Chord
4			1	T-BC	Case 5—Bottom Chord

TENSILE TESTS OF STRUCTURAL STEEL SPECIMENS

SPECIMEN	AREA		LB. PER SQ. IN.		PER CENT ELONGATION		REDUCTION OF AREA		FRACTURE
	Size	Sq. In.	Yield Point	Tensile Strength	2 In.	8 In.	Size	Per Cent	
Small Angles									
ID-TC A	1.000x.252	.252	44,500	68,200	37.0	23.5	.751x.137	59.2	Irregular Br. Silky.
ID-TC B	1.000x.248	.248	43,150	67,500	37.0	19.8	.735x.127	62.3	Angular Br. Silky.
ID-TC C	1.007x.247	.249	41,000	64,500	35.0	22.5	.792x.172	45.3	Angular Br. Silky.
ID-TC D	1.000x.245	.245	42,000	64,650	40.0	24.5	.729x.125	62.8	Angular Br. Silky.
1-BC A	1.005x.245	.246	39,850	61,750	34.0	19.7	.751x.127	61.2	Angular Br. Silky.
1-BC B	1.008x.250	.252	41,100	63,100	38.0	22.0	.748x.135	59.9	50-50 Cup and Cone-angular, Silky.
10-BC	1.010x.250	.252	38,500	59,700	38.0	22.9	.745x.135	60.1	Angular Br. Silky.
Large Angles									
WAMB-BC	1.395x.245	.342	40,200	62,250	46.0	26.4	1.03 x.122	63.1	50-50 Cup and Cone-angular, Silky.
ID-BC A	1.397x.237	.331	37,800	60,250	37.0	24.1	1.07 x.143	53.75	Angular Br. Silky.
ID-BC B	1.395x.245	.342	43,000	64,000	43.5	26.0	1.05 x.135	58.5	Irregular Br. Silky.
ID-BC C	1.385x.255	.353	42,100	63,450	43.0	25.0	1.03 x.134	60.9	Angular Br. Silky.
Large Angles									
1-TC A	1.572x.247	.388	41,500	65,500	42.0	26.1	1.19 x.123	63.9	Angular Br. Silky.
1-TC B	1.562x.247	.386	42,500	65,600	44.5	26.5	1.17 x.116	64.75	Angular Br. Silky.
1-TC C	1.570x.253	.387	42,600	67,600	41.0	24.1	1.20 x.137	58.7	Angular Br. Silky.
1-TC D	1.567x.253	.396	42,900	67,800	42.0	24.1	1.22 x.152	53.25	Angular Br. Silky.
3 in. Pipe									
T-TC A	1.005x.213	.214	33,200	55,000	44.5	29.5	.670x.100	68.7	Angular Br. Silky.
T-TC B	1.005x.212	.213	36,400	60,300	40.0	27.1	.714x.115	61.5	2/3 angular, 1/3 Cup and Cone, Silky.
2 1/2 in. Pipe									
T-BC A	.728x.207	.151	43,000	63,550	28.0	17.0	.510x.105	64.5	50-50 Angular, Cup and Cone.
T-BC B	.735x.207	.152	41,400	65,100	26.5	15.5	.515x.110	62.7	Angular Br. Silky.

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